ADDIS ABABA SCIENCE & TECHNOLOGY UNIVERSITY

COLLEGE OF ARCHITECTURE AND CIVIL ENGINEERING

POST GRADUATE STUDY

HYDRAULIC PERFORMANCE EVALUATION OF WATER SUPPLY DISTRIBUTION NETWORK

(THE CASE OF OLONKOMI TOWN, WEST SHOA ZONE, ETHIOPIA)

AN INDEPENDENT PROJECT SUBMITTED TO COLLEGE OF ARCHITECTURE AND CIVIL ENGINEERING POST GRADUATE STUDIES OF ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF ENGINEERING IN CIVIL ENGINEERING (HYDRAULIC ENGINEERING).

BY

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ADDIS ABABA, ETHIOPIA

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MARCH 1, 2017

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Abbreviations

GC	Gregorian Calendar
MDD	Maximum Day Demand
PHD	Peak Hour Demand
WSSE	Water Supply and Sewerage Enterprise
ICMM	International Council on Mining and Metals
WHO	World Health Organization
WDS	Water Distribution System
CSA	Central Statistics Agency
FCV	Flow Control Valve
GTP	Growth and Transformation Plan
L/c/day	Litter per Capital per day
UFW	Unaccounted for Water
m^3/d	Meter cube per day
L/s	Litter per second
m	meter
m^3	Meter cube
MOWR	Ministry of Water Resource
hl	head loss
m/km	meter per kilometre
m/s	meter per second
DN	Nominal Diameter
uPVC-	Polyvinyl chloride
DCI	Ductile Iron
GI	Galvanized Iron
HGL	Hydraulic Grade Line

EGL	Energy Grade Line
FDRE	Federal Democratic Republic of Ethiopia

Abstract

Hydraulic network analysis of water supply distribution system to address water distribution bottlenecks within an urban water supply system is important. This can be achieved through investigating the status of the existing distribution system of the network.

In this project the hydraulic performance of Olonkomi town is assessed using predefined hydraulic formula in spreadsheet.

The main objective of this study is to investigate the hydraulic performance of the water distribution system of the town.

The analysis was performed using predefined hydraulic formula in spreadsheet for average day demand and peak hour demand. After analysing the water distribution system results for allowable maximum pressure, minimum pressure and velocity was used as base to evaluate the hydraulic performance.

From analysis result, it is observed that there are different problems in the system. These are aged pipes, undersized pipes, low pressures and low velocity. This problem has been solved by replacing the aged pipes with new one and using the design criteria of velocity and pressure for undersized pipes, low pressure and low velocity.

Finally, 44% of the total distribution pipe lines need modification currently at the base year which are smaller diameter and above its service life of steel pipe and should be replaced with newer one.

Chapter1. Introduction

1.1 Background

Water is a fundamental resource for life. Whether from groundwater or surface water sources, availability of water and access to water that meets quality and quantity requirements, is a critical need across the world. However, factors such as population growth and economic development mean that its availability is becoming increasingly constrained in many areas. Although water issues are important globally, they are first and foremost local issues and always particular to specific areas. Areas where there is not enough water to meet the demand for water are considered to be areas of “water stress”. The availability and demand may be different even within short geographic distances [6].

In Sub-Saharan Africa access to water supply and sanitation has improved, but the region lags behind all other developing regions. There are large disparities amongst countries in the Sub-Saharan region[16].

Like other African countries, in Ethiopia, shortage of water supply is also observed due to increased population, expansion of industries and economic development. Since the problem of water supply can result in social, political and economic problem on the society, measures should be needed to supply potable water in adequate quantity for the consumers.

Today, apart from supply and demand gap, water distribution modelling is a critical part of operating water distribution systems that are capable of serving communities reliably, efficiently, and safely, both now and in the future [10]. Even if water production is improving from time to time in Olonkomi town, the present situation of water distribution is characterised by an insufficient supply with low pressure, low flow and unacceptable high rate of leakage and pipe failure due to unmaintained minimum and maximum pressure in the distribution system which increase water shortage within distribution system.

Therefore, analysis of a pipe network is essential to understand or evaluate a pipe network system that ensure sufficient pressure and flow at the point of supply within a range whereby the maximum pressure avoids pipe bursts and the minimum ensures that water is supplied at adequate flow rates for all expected demands.

Since the population in the town is increasing and burden on the water distribution system, looking the hydraulic performance of the water distribution network of pipes is necessary to solve the problem of water distribution system.

1.2 Existing Water Distribution System

According to the data obtained from Olonkomi town water supply and sewerage services, the town source of water supply, which is currently functional, is from ground-water source of two deep wells. The first well which yields about 3 l/s was drilled by Oromia Water, Mineral and Energy Bureau in 2006 and the second well, which yields about 9.3 l/s was also drilled by Oromia Water, Mineral and Energy Bureau in 2010.

The water supply system components include main transmission pipes from bore hole to reservoir, $50m^3$ and $150m^3$ masonry sandwich reservoirs, distribution pipelines, two wells, pump stations, 13 public water points and control valves.

1.3 Statement of the Problem

Inadequacies of water supply in a distribution system are the major problems facing water industry over the world[13]. The problem is severe in most developing-country including Ethiopia; where increased urbanization, population growth, poor city planning, and shortage of sufficient resources creating combined effect[16].

Rapid urbanization rate associated with very high population flow to the town from rural is expected as the town is newly established town of Ejersa Lafo district with high potential development land for industry, residential houses and other social economic and financial infrastructures.

The potential water resources of Olonkomi town are surface water sources and ground water sources. The surface water sources comprises of three rivers, Kela river in the east, and Jemjem and Cheleleka rivers in the west of the town. All the rivers are perennial and are flowing towards south of the town and join Awash river. The ground water source is boreholes from which currently the town uses as a water supply sources, drilled boreholes, located along the Jemjem river.

According to the data obtained from Olonkomi town water supply and sewerage Authority, the quantity of water production of the wells currently are 131679 m^3/year which means around 360m^3/day.

Presently Olonkomi town, the town of Ejersa Lafo district, faces a serious problem of water supply to deliver the require quantity of water for the consumer's with sufficient pressure and flow at the point of supply.

Therefore, to have sustainable water supply systems for the consumers, hydraulic analysis using predefined hydraulic formula in spreadsheet is selected to evaluate the hydraulic performance of the water distribution network and improving its performance to meet demands from new developments and increased consumption.

1.4 Objectives

1.4.1 General Objective

The main objective of this study is to investigate the hydraulic performance of the water distribution system of Olonkomi town, Ejersa Lafo District.

1.4.2 Specific Objectives

- To examine the hydraulic performance of existing water supply distribution network using velocity and pressure parameters ;
- To check whether the quantity of water harvested from wells are sufficient or not;
- To recommend remedial measures for the water supply distribution network problems.

Chapter 2 Literature Review

2. 1 Water Distribution System

Water distribution system is a network of pipe lines inside the municipal limit, for transporting treated water to the consumer [5]. The water may be supplied for different kinds of uses such as domestic, commercial, industrial agricultural and public. In general, water distribution systems can be divided into four main components [11]:

(1) Water sources and intake works,

(2) Treatment works and storage, `

(3) Transmission mains,

(4) Distribution network.

The common sources for the untreated or raw water are surface water sources such as rivers, lakes, springs, and man-made reservoirs and groundwater sources such as bores, wells, and developed springs. The intake structures and pumping stations are constructed to extract water from these sources. The raw water is transported to the treatment plants for processing through transmission mains and is stored in clean water reservoirs after treatment. The degree of treatment depends upon the raw water quality and finished water quality requirements. Sometimes, groundwater quality is so good that only disinfection is required before supplying to consumers. The clean water reservoir provides a buffer for water demand variation as treatment plants are generally designed for average daily demand. Water is carried over long distances through transmission mains. There are no intermediate withdrawals in a water transmission main. A distribution network delivers water to consumers through service connections.

2.1.1 Objectives of Water Distribution System

Water distribution system has the following objectives [2]:

- To convey the water to point of supply from the treatment plant.

- To preserve the water quality from treatment up to the consumer end.
- To ensure sufficient pressure and discharge at all places during all times.
- It must be capable of meeting the emergency demand of fire fighting

2.1.2 Methods of Distribution Systems

The main purpose of the distribution system is to develop adequate water pressure at the consumer taps. The choice of the distribution system depends upon the topography of the area of distribution and the elevation with respect to the location of the water treatment plant. The distributions systems may be classified in three categories [5].These are gravity system, pumping system without storage and dual system with storage (combination of gravity and pumping system).

In the gravity system, the elevation of the source of supply in relation to the area of distribution is kept such that adequate water pressure in the pipes at different points is available. In this system pumping is normally not required. However, if water sources (lake, dam, reservoir etc.) are used as the source of water is behind the hill and water purification unit is situated on the hill itself, then water may have to be pumped from the sources to the purification plant. But purified water flows to the distribution system without pumping. This is the most reliable and economical method of water distribution.

In pumping system without storage, purified water is directly pumped in to the distributing mains for obtaining the required pressure. It is the most undesirable system, because power failure would mean complete interruption in the water supply. Also, since the consumption varies from time to time and from hour to hour, the pressure in mains will keep on fluctuating. The pumps will have to be run at varying speeds according to the variation in the consumption and thus necessitate constant attendant on pumps. Pumps also wear out in very short time.

On the other hand, in dual system with storage (combination of gravity and pumping system), the excess of water pumped during period of low consumption is stored in elevated tanks. As the time of high consumption the stored water in the elevated tanks augments the pumping and peak demand is fulfilled. Pumps have not to be run at varying speeds but at constant speed, thus reducing the wear of the pumps. This method is more reliable and economical. Stored water in elevated tanks also fulfils water requirements for sometimes during break down of the pumps, and for fire fighting.

2.1.3 Systems of Supply of Water

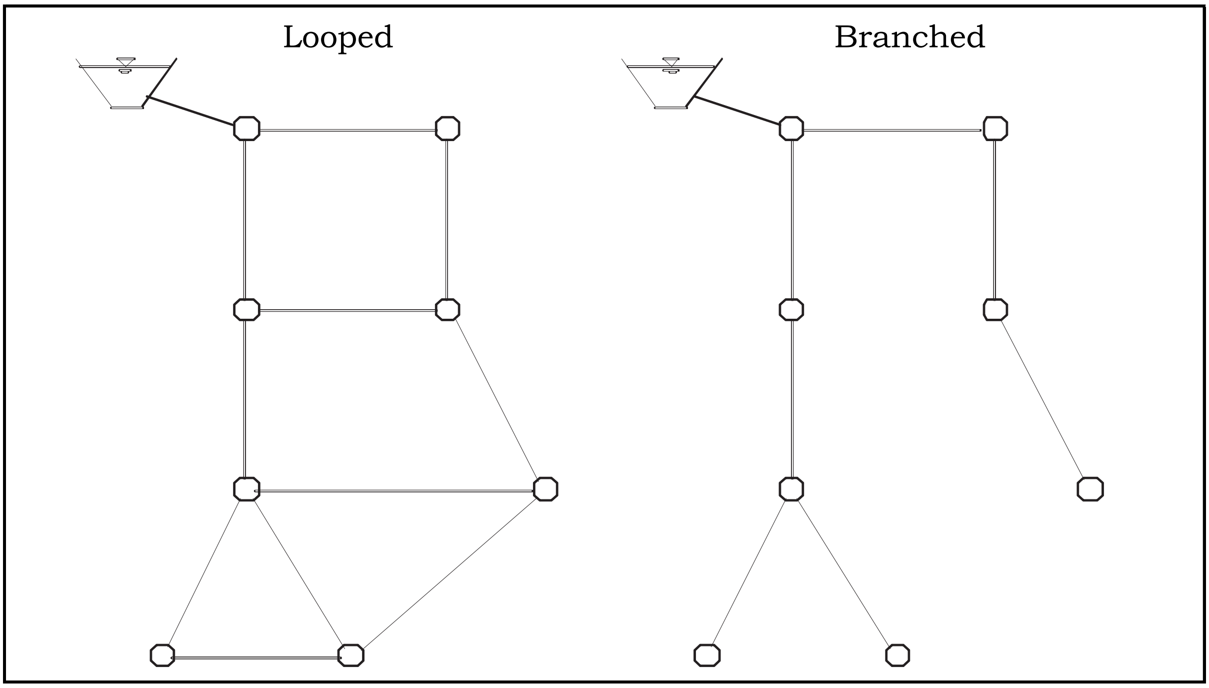
Water may be supplied to the consumers by the continuous and intermittent system. In continuous system of supply; water is supplied to the consumers for all the 24 hours of the day. This is the most ideal system of supply of water and it should be adopted as far as possible. The disadvantage of this system is that considerable wastage of water occurs if there are some leakages and also if the consumers do not realise the cost of treated water. In the intermittent system of supply, water is supplied to the consumers during certain fixed hours of the day only. In this system of supply, the distribution area is divided in to several zones and timings for the supply of water to each zone are so adjusted that good working pressure is maintained in each zone. The intermittent system of supply of water is useful when the quantity of water available is not sufficient to meet the various demand of water and the available pressure is poor.

On the other hand, the intermittent system of supply of water has several drawbacks. Some of them are, the consumers have to store water for the non-supply period, which is likely to get contamination; fire extinguishing is not possible in non-supply period which cases huge loss of property and human beings; there is wastage of water as the taps may leave open in search of water during non-supply hours; There may also wastage of water because the stored water if not used will have to be thrown off to store the fresh water; large number of valves and extra staff will be required to operate and maintain these valves; during non-water supply period the emptying mains tends to create vacuum, which allows the infiltration of polluted sub-solid water through leaky and defective joints [5].

2.1.4 Layout of Distribution System

In the water distribution networks the street patterns, topography, construction plans and future plans determine the layout of pipes. The water distribution networks have mainly the following three types of configurations: Branched or tree-like configuration, looped configuration and branched and looped configuration [11].

A branched network, or a tree network, is a distribution system having no loops. A pipe network in which there are one or more closed loops is called a looped network. Looped networks are preferred from the reliability point of view. If one or more pipelines are closed for repair, water can still reach the consumer by a circuitous route incurring more head loss. On the other hand, the branched pipe networks do not permit the water circulation since they contain lots of dead ends. Furthermore, if a pipe repair is needed the whole branch cannot deliver water in branch systems. In real life networks, it is very hard to have a totally looped system. Most of the water distribution systems are a combination of looped and branched systems.



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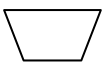


Reservoir

Pipe line

Junction/node

Source: Thomas M., *et. al*, 2003

Figure 2.1 Looped and Branched Networks

2.1.5 Water Distribution Network Elements

The following brief explanations about water distribution network elements and pipe line materials selection were taken from[13].

Moving water from the source to the customer require a network of pipes, pumps, valves, and appurtenances. Storing water to accommodate fluctuations in demand due to varying rates of usage or fire protection requires storage facilities such as tanks and reservoirs. Piping, storing and supporting infrastructure are together referred to as the water distribution system.

2.1.5.1 Pipes

Pipes are mostly a circular conduit in which water flows under pressure. It is the main components of water distribution systems. They can be found in different lengths, materials and diameters laid down in the network. The pipes are mainly grouped into three:

- Transmission pipes
- Distribution pipes
- Service pipes

The transmission main line is the pipe between the source and the storage elements; it carries water from source or pump station to the storage tank.

Rising and gravity transmission mains from source to distribution should be designed for the maximum day demand, based on the design hours of water source operation. The number and diameters of transmission pipes should be determined primarily on the basis of economic considerations, comprising either a single large diameter pipe of sufficient capacity for the final planning horizon or several parallel pipes of smaller diameter, installed at various intermediate horizons. The economic analysis should take into account the cost of pipe and energy to determine its optional diameter (which should normally be selected from the standard range diameters). However, engineering considerations should also be taken into account if important [8].

Where transmission or gravity mains involve working or static pressures that are higher than advisable in relation to pump capacities or pipe pressure ratings, and then break pressure tanks and/or booster stations are considered.

No house connections should be made to transmission mains [9].

The design of transmission mains in water supply systems should have to address the following design activities:

• Sizing for ultimate future design flows;

• Sizing and layout to ensure adequate supply and turnover of water storage

facilities;

• Elimination of customer service take-offs;

• Minimization of branch take-offs to help maintain flow and pressure control

• Air relief at high points and drain lines at low points;

• Isolation valve to reduce the length of pipe required to be drained in a repair or maintenance shut-down;

• Potential transient pressures; and

• Master metering.

Normally, the sizing of the transmission main is dependent on the total storage capacity and the way the supply is transmitted to the distribution system. For direct pumping to the distribution system with no reservoir, the transmission main is designed for a maximum carrying capacity equivalent to the peak- hour demand. For systems with a storage reservoir, the transmission main to the reservoir is designed at a carrying capacity of maximum day demand.

The main should have at least the carrying capacity to supply water at a rate equivalent to the maximum day demand of the system for a given design year [17].

In cost effective design, for towns where the minimum standards for clean water supply are not yet met, the service level in terms of quantity of flow would be the minimum standard value adjusted for the Maximum Day demand. Higher service levels require large volume of water to be transported by the transmission line, which implies larger diameter pipe. Cost effective size of the transmission line can be obtained by selecting a service level that is affordable to the community and designing the sizes to the affordable quantity of flow adjusted for the maximum day flow.

The design of all transmission mains shall be correlated with projected supplies from the source facilities and storage. In specific cases involving long transmission mains delivering pumped water, an economical evaluation shall be made on costs of various size transmission mains versus pumping costs.

The sizing of all transmission mains shall take into consideration the minimum pressure specified.

It is preferable that flow velocities remain in the range of 0.6 to 1.5 m/s although in exceptional conditions this can rise to approx. 2.0 m/s [8].

As a rule of thumb, for transmission by pumping, it is advisable to assume a preliminary head loss (hL) of about 5.0 m/km of pipeline. (As much as possible, head loss should be limited to10.0 m/km of pipeline for transmission by pumping.) For a gravity system with a considerably elevated source (e.g. highland springs), the transmission line could afford to have higher head losses as long as the remaining pressure head at the downstream end is sufficient for the distribution system's needs. For a gravity system with source that is not much higher than the distribution system, the head losses are lowered to attain sufficient pressure head in the distribution system [2].

The pipe material for transmission main must be selected to with stand the highest possible pressure that can occur in the pipeline.

For the transmission line design, a maximum computed HGL based on a minimum supply rate, maximum day demand should be examined. At any point in the transmission line, this maximum HGL should not be over the allowable maximum pressure of the line (70 m head).

To limit the maximum pressure, break pressure tanks or chambers could be installed along the main. The break pressure tank or chamber will limit the static pressure by providing an open water surface at certain points of the transmission line [17].

A distribution network delivers water to consumers through service connections. Such a distribution network may have different configurations depending upon the layout of the area. Generally, water distribution networks have a looped and branched configuration of pipelines, but sometimes either looped or branched configurations are also provided depending upon the general layout plan of the city roads and streets [9].

Water distribution systems are made up of pipe, valves, and pumps through which treated water is moved from the treatment plant to domestic, industrial, commercial, and other customers. The distribution system also includes facilities to store water, meters to measure water use, fire hydrants and other appurtenances. The major requirements of a distribution system are to supply each customer with sufficient volume of treated water at an adequate service pressure [17].

The distribution network will be designed for the peak hourly demand. The minimum pipe size to be considered for primary and secondary networks should be nominal diameter of 2 inch /DN 2”/. Tertiary pipes may be below DN 2”, but not below DN 1”. Large scale networks may conceivably have a larger minimum diameter for primary and secondary pipes. Distribution systems should be planned with either one large diameter pipe suitable for the final planning horizon, or multiple smaller diameter pipes installed at various intermediate-planning horizons. An economic analysis should be carried out to determine the cheapest solution [9].

Generally; the static state pressures in pipelines must be less than the pipe nominal pressure rating. In the case of long mains where water hammer risk is expected, due attention must be given to the pipe material and a proper water hammer analysis carried out [9].

It is preferable that flow velocities remain in the range of 0.6 to 1.5 m/s although in exceptional conditions this can rise to approx. 2.0 m/s [8].

On the other hand, service pipes are the pipes that mainly deliver water to the consumers.

2.1.5.2 Pumps

A pump is a hydraulic machine that adds energy to the water flow by converting the mechanical energy into potential energy to overcome the friction loses and hydraulic grade differentiations within the system.

The pump characteristics are presented by various performance curves such as, power head and efficiency requirements that are developed for the friction rate. These curves are used in the design stage to find out the most suitable pump for the system. In most of the pumping stations two or more pumps are used to ensure reliability, efficiency and flexibility. Pump efficiency plays an important role in water distribution network management as a high percentage of total expenses are used for their electricity or fuel bills.

2.1.5.3 Valves

There are different types of valves in water distribution systems with different characteristics and usage conditions. Their locations and characteristics are significant for the daily management.

Check Valves

Check valves are the valves that prevent the water flow backwards from the desired direction. It is the valve only allows flow in one direction. When water flows in the direction of need, check valve status is open; on the other hand, when the flow changes its direction, the check valve's status is automatically closed in order to permit the flow. They are widely used in front of the pumps in order to prevent reverse water flow through the pumps.

Flow Control Valves (FCV)

Control valves are used to limits the flow rate through the valve to a specified value, in a specified direction. It is commonly used to limit the maximum flow to a value that will not adversely affect the provider's system. Generally butterfly types of valves are used for that purpose. These types of valves generally used for regulating purposes and controlling the overall pressure on the sub-pressure zones.

Isolating Valves

When a pipe breaks or if a maintenance work is needed, in order to isolate the pipe or pipe segment from the rest of the network, isolating valves are used. Generally gate valves are chosen as isolating valves. Despite of control valves, their ability to control the flow is very limited. For that purpose, the isolating pipes should be used in the fully close or open position, as partially open valves may end with broken valves in the system.

Furthermore, isolating valves are the mostly used valves in a network. Their locations and working conditions directly affect the distribution systems characteristics and reliability purposes.

Air Release Valves

Air in the water distribution system must be taken out from the network in order to have system stable. For that purposes, air release valves are used. Valve will begin to open when pressure in the pipeline exceed a set pressure (determined by force on the spring).They are usually located at the high points of pipes as mostly air is trapped and purged at these locations.

Pressure Reducing Valves

Pressure reducing valves are the valves that used to prevent the high inlet pressure pass through the outlet. As the water flows from pressure reducing valve, the pressure is reduced to the desired level by proper adjustment of the valve. These types of valves are generally used in between the zones with high elevation differences. Furthermore, these valves have the flow controlling abilities.

Sluice Gates

Are vertically sliding valves which are used to open or close openings in to walls [10].

- **Fire hydrants.** It is used on mains to provide a connection for fire hazards to fire fighting
- **Water meters.** Measure the water carried from borehole, supplied to the reservoir, out from reservoir and furnished to a consumer, and the consumer charged accordingly to the amount of water consumed.

2.1.5.4 Storage Tanks

The main purpose of a storage tank is to store excess water during low demand periods in order to meet widely fluctuating demands such as fire demands and peak hour's demands.

A storage tank's oscillations are directly integrated with the demand and pump working rate. Generally tanks are used as distribution reservoirs to supply coming from the pump and store the excess flow during night. Another usage of storage tank is that they stabilize the excess pressure over the network by opening the system to the atmospheric pressure.

The volume of storage tank is determined via a mass flow balance. Data for mass flow balance analyses would need demand pattern study besides operational schedules. The hourly demand factors is the basis to determine the volume of the reservoir. In the absence of hourly peak factor data, the volume of the service reservoir can be determined taking 8 –12 hrs. of the average day demand [8].

2.1.5.5 Public Taps

Public taps should be installed to provide a maximum walking distance of 500 m in any direction in town's to obtain access of potable water. The definitive spacing and location of public taps should be determined in collaboration with the served community taking into consideration the operating hours and the number of faucets per installation. Locations should be fixed during the design stage or during the construction stage if such details are left open during the design. Supply pressures at public taps should be limited to a range of 2 to 5 metres using a suitable pressure reducing valve [9].

2.2 Network Analysis of Water Distribution System

Water distribution network analysis provides the basis for the design of new systems and the extension of existing systems. Design criteria are that specified minimum flow rates and pressure heads must be attained at the outflow points of the network [5]. Accordingly in networks of interconnected hydraulic elements, every element is influenced by each of its neighbours; the entire system is interrelated in such a way that the condition of one element must be consistent with the condition of all other elements.

The basic principles governing network hydraulics are [14]:

•Conservation of mass – the fluid mass entering any pipe system will be equal to the mass leaving the system. In network analysis, outflows are lumped in nodes. A related principle is that at each junction (node), the algebraic sum of the quantities of water entering and leaving the node is zero.

•Conservation of energy – In any closed path or circuit in a hydraulic network, the algebraic sum of the energy (head losses) in the individual pipes is zero. Another way of stating it is that the difference in energy (head loss) between two nodes in a system must be the same regardless of the path that is taken (Bernoulli's principle).

2.3 Sources of Water Supply

The primary source of water is precipitation (rain fall), which may be available in the form of surface water or ground water [2].

1. Surface-water: perennial stream, lakes, rivers and canals with adequate flow are considered reliable sources of water supply for town or a city. Excessive flood water is stored by constructing impoundments across rivers for use, during the lean period(deficit period)
2. Ground-water: Ground water is tapped from aquifers for public or private use through wells, springs and infiltration galleries. The yield depends on the depth, type of aquifer and ground water table gradient. Good yielding of aquifers can also be considered as reliable sources of water supply for community purposes.

2.3.1 Factors Governing the Selection of Source of Water

The following important factors are generally considered in selection of a particular source for supplying water to a city or a town[10].

(i) **The Quantity of Available Water**: the quantity of water available at the source must be sufficient to meet the various demands during the entire design period of the scheme. If sufficient quantity of water is not available in the vicinity of the area, we may have to think of bringing water from distant sources.

(ii) **The Quality of Available Water:** the water available at the source must not be toxic, poisonous or in any other way injurious to health. The impurities present in the water should be as less as possible, should be removed easily and economically by normal treatment methods.

(iii) **Distance of the Source of Supply:** the source of water must be situated as near the city as possible. Because when the distance between the source and the city is less, lesser length of pipe conduits and other associated appurtenances are required less, thereby reducing the cost.

(iv) **General Topography of the Intervening Area:** the area or land between the source and the city should not be highly uneven i.e. it should not contain deep valleys or high mountains and ridges. In such uneven topographies the cost of supports and joints for carrying water pipes in valleys and that of constructing tunnels in mountains shall be enormous.

(v) **Elevation of the Source of Supply:** the source of water must be on a high contour, lying sufficiently higher than the city or town to be supplied with water, so as to make the gravity flow possible. When the water is available at lower levels than the average city level, pumping has to be resorted to, which involves huge operational cost and frequent possible breakdowns.

2.3.2 Quality of Source of Water Supply

The water used for drinking purpose should be free from impurities like iron, manganese, nitrate, calcium, magnesium and chlorine or contain them in permissible limits [2].The following are the requirements of potable water for domestic use [5]:

(i) It should be free from disease producing bacteria.

(ii) It should be colourless, odourless and clear.

(iii) It should be testy

(iv) It should not corrode pipes and other fittings.

(v) It should be free from harmful salts and other objectionable matter.

(vi) It should be free.

2.4 Pipe Line Materials Selection

2.4.1 Factors in Selecting Pipeline Materials

The following are factors to be considered in selecting pipeline materials[1]:

- Flow Characteristics: The friction head loss is dependent on the flow characteristics of pipes. Friction loss is a power loss and thus may affect the operating costs of the system if a pump is used.
- Pipe Strength: Select the pipe with a working pressure and bursting pressure rating adequate to meet the operating conditions of the system. Standard water pipes are satisfactory usually only in low pressure water supply systems.
- Durability: Select the type of pipe with good life expectancy given the operating conditions and the soil conditions of the system.

- Type of Soil: Select the type of pipe that is suited to the type of soil in the area under consideration. For instance, acidic soil can easily corrode G.I. pipes and very rocky soil can damage plastic pipes unless they are properly bedded in sand or other type of material.
- Availability: Select locally manufactured and/or fabricated pipes whenever available.
- Cost of Pipes: Aside from the initial cost of pipes, the cost of installation should be considered. This is affected by the type of joint (such as screwed, solvent weld, slip joint, etc.), weight of pipe (for ease of handling), depth of bury required, and width of trench and depth of cover required.

2.4.2 Types of Pipes Used in Water Supply Systems

Pipes found in waterworks systems are generally of the following materials [1]:

- Ductile Iron (DI);
- Steel;
- Polyethylene (PE);
- PVC (Polyvinyl chloride);
- GRP(Glass reinforced Plastic);
- Pre-stressed concrete, cylinder or non-cylinder(PSC);
- Reinforced concrete cylinder(RC);
- Asbestos cement
- Galvanized iron, copper and lead

2.5 Economic Lives of Water Supply System Components

The following service lives for system components was adopted for economic analysis calculations:

Table 2.1 Service Lives for Water Supply System Components

System Component	Years
Boreholes in hard rock	25
Boreholes in limestone	15
Electromechanical equipment of pumping stations and boreholes	10
Ductile iron pipes	40
PVC pipes	25
Steel pipes	30
Masonry/Solid block water tanks	25
Concrete works	50
Concrete water tanks	50
Civil engineering building works (general)	40
Treatment plants	50
Chemical dosing	10

Source: Ministry of Water Resource of Ethiopia, 2006

2.6 Design Period

It is the period for which the water supply schemes are designed to serve over a specified period of time after completion of the project [2]. The design period has a direct impact on the overall capacity, complexity as well as cost of water supply systems [8]. During this period the components, structures and equipment's of the project are supposed to be adequate to serve the requirements [2].

The following are factors that affect the design period [2]:

- Useful life of the pipes, structures and equipment used in the water works. If the useful life of materials is long, design period is also long.

- The anticipated rate of growth of population. If the rate is high, design period is short.
- The rate of interest of loans taken for the construction of the project. If this rate is high the design period will be short.
- The rate of inflation during the period of repayment of loans. When the inflation rate is high, a longer design period is adopted.

2.7 Population Projection

Population projection is very important in any water supply project in order to determine the future water requirements of the consumers and to make the system sufficient. Hence, the planning of any water supply system has to be based on the forecast of population size, population growth rate and distribution [6].

There are a number of factors that should be taken in to consideration in projecting the future population size of a project, some of which are fertility, mortality, economic activity in the project area, availability of natural resources, and status of the village, i.e. its economic significance, relative location of the Village with respect to main highways and availability of reliable urban infrastructure facilities and etc. [16].

The following are the methods used for population forecasts [5]:

1. Arithmetical Increase Method. In this method, the increase in population is assumed to be constant. An average increment in the population of the past three or four decades is worked out. This method underestimates the rate. This method can be adopted for forecasting population of large cities which have achieved saturation condition.

$$P_n = P + nI \text{ ---------------------------- } Eq.2.1$$

where

P_n= Future population

I= Average increase for last two or three decades

P=Present population of a particular town

n= Number of decades

2. Geometrical Increase Method or Uniform Percentage Growth Method. This method assumes the percentage increase in population from decade to decade as constant. This method gives high results. The percentage increase gradually drops when the growth of the city reaches the saturation point.

$$P_n = P(1+\frac{IG}{100})^n -------------------------Eq.2.2$$

Where

P_n= Future population

P= Present population of a city

IG= Average percentage increase per decade

n= Number of decades

3. Incremental Method or Method of Varying Increment. This method embodies the advantages of the earlier two methods. The average of the increase population is taken as per arithmetical method and to this, is added the average of the net incremental increase, one for every future decade whose population figure is to be estimated. In this method, a progressive increasing or decrease rate is adopted rather than constant rate

$$P_n = P + nI + n(\frac{n+1}{2})r --------------------Eq.2.3$$

Where

P= Present population

P_n = Population at the end of n future decades

r=Average incremental increase

r=<u>Net incremental value per decade</u>

n

n= Number of decades

I= Average increase for last two or three decades

4. Graphical Extension Method. In this method, a curve is plotted between past population, and corresponding census year. This curve is then extended to the cover the design period of the water supply scheme, and the population after each successive future decade is read out from the curve.

5. Logistic Curve Method or S-Curve Method. The rate of increase of population of a city never remains constant. The growth of new city is very slow in the beginning. After a certain minimum level of growth, the population of the city grows by a very high rate and lastly rate of growth progressively lowers down till a saturation limit of population reached. The saturation limit of population depends upon the limit of economic opportunities which the city can provide. Thus if population of the city is plotted against the year of its growth, for the full time of it follows a S-shaped curve. This curve is known as s curve or logistic curve.

6. Graphical Comparison Method. In this method, cities having similar conditions and characteristics, to the city whose population is to be estimated, are selected. It is presumed that all these cities had grown under similar conditions. The rate of increase in population in comparable cities is applied to the city under consideration for estimating its future population.

7. Zoning Method. This is the most reliable and useful method of population forecast. In this method, master plan of the city for its future development is prepared. This master plan is divided in to several zones, such as industrial, commercial, and residential zones and the city is allowed to develop as per master plan only. When all the zones are fully developed the future population can be worked out easily.

8. Ratio and Correlation Method. This method of forecasting population is based upon the fact that population of the cities or other areas have a direct relationship to the population of the whole country. Therefore, it is possible to forecast the population of the city under question by considering the rate of population growth of the country as a whole.

9. Growth Composition Analysis Method. This method depends up on the determination of the rate of births, deaths, and migration tendencies.

2.8 Estimation of Water Demand

The design and execution of any water supply scheme requires an estimate of the total amount of water required by the community [2].

The annual average demand for water, i.e. per capita demand, considerably varies for different towns. These variations in total water consumption depends upon various factors, which must be thoroughly studied and analysed before fixing the per capita demand for design purposes.

The following are the common factors which affect the rate of demand of water [5].

Climatic Condition: Climatic condition has great influence on water consumption rate. The amount of water requirements in hot and arid regions will tend to increase as compare to wet and cold places

Standard of Living/People's Habits: Rich people with a high standard of living require more water than those belonging to the middle class and low – income groups

Cost of Water: The rate at which water is made available to the consumers may also affect the rate of demand. The more costly is the water, the lesser will be the rate of demand.

Quality of Water: There is high consumption of water if the quality is good enough as people consider it safe for their life; otherwise there should be less consumption. Similarly, certain industries which require certain standard quality waters will not develop their own supplies and will use public supplies.

System of Sanitation: Cities or towns having sanitation water carriage system of drainage will consume more water as water will be required for flushing sanitary units; such as urinals, water closets, etc.

System of Supply: For the cases of continuous and intermittent water supply systems, it is definitely true that much water should be necessary in the continuous flow system.

Use of Meters: Meters fitted on the mains supplying water to the houses record the quantity of water supplied to the consumers. In this case consumers have to pay as per quantity of water supplied to them and everybody should be careful due to sense of economy developed in them etc.

2.9 Variations in Rate of Consumption

The annual per head daily water demand does not remain constant throughout the year. The demand variation is dependent on the consumption pattern of the locality. It varies from season to season and day to day. Even in a day there is variation in demand from hour to hour.

Variation in rate of demand may be classified as:

i). Seasonal variation: - The rate of demand of water keeps on changing from season to season. In hot season, more water is consumed for drinking, bathing and washing clothes than in wet season.

ii). Daily variation:- The rate of demand for water may vary from day to day . The consumption of water is high at weekends and holidays than on normal days. It is due to climatic conditions and also due to holidays.

iii) Hourly variation:- Demand of water, during 24 hours of the does not remain constant. It varies according to hour of the day. Peak demand occurs in the morning and evening than early in the afternoon and late at night [5].

Thus, the annual average daily consumption, while useful, does not give the complete picture. Therefore, to account these fluctuating water demands, it is necessary to step up the average day demand by certain factor to get the maximum day demand and the peak hour demand. These scaled up water demand figure are used for planning and design of water supply systems.

2.10 Effects of Demand Variation on the Design of Water Supply Scheme Components

The various units involved in water supply schemes should be designed not only to serve the average daily demand but also to serve the maximum demand arise and also the variations in the demand [4]. The following recommendations may be adopted for designing the capacities of different components [4]:

- The source of supply, transmission mains, pumps and service reservoir are designed for maximum daily demand.
- The distribution system: it should be designed for peak hourly demand i.e. to deliver the peak water demand during the peak hour of the day.

2.11 Pipeline Hydraulics

2.11.1 Pressure

Pressure is a force applied perpendicular to a body that is in contact with a fluid.

The pressure exerted by a column of water is called pressure head, and can be calculated using the formula below:

$$h = \frac{P}{\gamma} ------------------------------ Eq.2.4$$

Where,

h=Depth of water above a datum (m)

P=Pressure (p_a)

γ=ρg =Specific weight of water (kg/ms)

2.11.2 Head Losses in Pipes

Head loss is the reduction in the total head or pressure (sum of elevation head, velocity head and pressure head) of the fluid as it moves through a fluid system. It is presented because of: the friction between the fluid and the walls of the pipe; the friction between adjacent fluid particles as they move relative to one another, and the turbulence caused whenever the flow is redirected or affected in any way by such components as piping entrances and exits, pumps, valves, flow reducers, and fittings[13].

In flow through piping systems, there are two types of head losses.

Major losses are that part of the total head loss that occurs as the fluid flows through straight pipes.

Minor losses are those due to any other “devices” in the piping system other than constant-diameter pipe sections. These include Pipe entrance or exit, sudden expansion or contraction, bends, tees, valves and other fittings etc.

2.11.2.1 Factors that Affect Head Loss

The following are factors that affect head loss [10]:

Flow Rate: When the flow rate increases, the velocity of the liquid increases, at the same rate. The friction or resistance to flow (due to viscosity) also increases. The head loss is related to the square of the velocity so the increase in loss is very high.

Inside Diameter of the Pipe: When the inside diameter is made larger, the flow area increases and the velocity of the liquid at a given flow rate is reduced. When the velocity is reduced there is lower head loss due to friction in the pipe. On the other hand, if the inside diameter of the pipe is reduced, the flow area decreases, the velocity of the liquid increases and the head loss due to friction increases.

Roughness of the Pipe Wall: As the roughness of the inside pipe wall increases so does the thickness of the slow or non-moving boundary layer of liquid. The resulting reduction in flow area increases the velocity of the liquid and increases the head loss due to friction.

Corrosion and Scale Deposits: Scale deposits and corrosion both increase the roughness of the inside pipe wall. Scale build up has the added disadvantage of reducing the inside diameter of the pipe. All of these add up to a reduction in flow area, an increase of the velocity of the liquid, and an increase in head loss due to friction.

Viscosity of the Liquid: The higher the viscosity of the liquid is, the higher the friction is from moving the liquid. More energy is required to move a high viscosity liquid than for a lower viscosity liquid.

Length of the Pipe: Head loss due to friction occurs all along a pipe. It will be constant for each SI unit of pipe at a given flow rate.

Fittings: Elbows, tees, valves, and other fittings are necessary to a piping system for a pump. It must be remembered that fittings disrupt the smooth flow of the liquid being pumped. When the disruption occurs, head loss due to friction occurs. At a given flow rate the losses for the fittings will be calculated using a factor that must be multiplied by a velocity head figure, or as the head loss equivalent to a straight length of pipe.

Straightness of the Pipe: Because of momentum, liquid wants to travel in a straight line. If it is disturbed due to crooked pipe, the liquid will bounce off of the pipe walls and the head loss due to friction will increase. There is no accurate way to predict the effects since "crooked" can mean a lot of things.

2.11.2.2 Formulae for the Calculation of Head Loss in Pipes

There are several formulae for the calculation of head loss in pipes which have been and are still used for the design of water supply systems. The commonly used formulas for computation of head loss due to friction (also called friction loss) are the:

•Darcy-Weisbach formula

$$h_f = \frac{fLV^2}{2gD} ------------------------ Eq.2.5$$

Where, hf = Head loss due to friction (m)

f = Friction factor (which is related to the relative roughness of the pipe material & the fluid flow characteristics)

L = Distance between sections or length of pipe (m)

V = Velocity of flow (m/s)

D = Internal diameter of pipe (m)

g = Acceleration due to gravity (m/s^2)

•Hazen-Williams formula

$$hL = \frac{10.7xLxQ^{1.852}}{C^{1.852}xD^{4.87}} ----------------------- Eq.2.6$$

Where, hL = head loss due to friction (m)

L = Distance between sections or length of pipe lines (m)

C = Hazen-Williams C- Value

D = Internal diameter of pipe (m)

Q = Pipe line flow rate (m^3/s)

•Manning's formula

$$Q = \frac{AR^{2/3}S^{1/2}}{n} ----------------------- Eq.2.7$$

Head losses also occur at valves, tees, bends, reducers, and other appurtenances within the piping system. These losses, called minor losses, are due to turbulence within the bulk flow as it moves through fittings and bends [14].

Head loss due to minor losses can be computed by multiplying a minor loss coefficient by the velocity head [12].

$$h_m = k_L \frac{V^2}{2g} ----------------------- Eq.2.8$$

Where,

h_m =Head loss due to minor losses (m)

K_L=Minor head coefficient

V=Velocity (m/s)

g = Acceleration due to gravity (m/s^2)

2.11.3 Energy Concepts

Fluids possess energy in three forms. The amount of energy depends upon the fluid's movement (kinetic energy), elevation (potential energy), and pressure (pressure energy). In a hydraulic system, a fluid can have all three types of energy associated with it simultaneously. The total energy associated with a fluid per unit weight of the fluid is called head. The kinetic energy is called velocity head ($V^2/2g$), the potential energy is called elevation head (Z), and the internal pressure energy is called pressure head (p/γ) [14].

$$H = Z + \frac{p}{\gamma} + \frac{V^2}{2g} ---------------------- Eq.2.9$$

Where,

H=Total Head (m)

Z= Elevation above datum (m)

P=pressure(N/m^2)

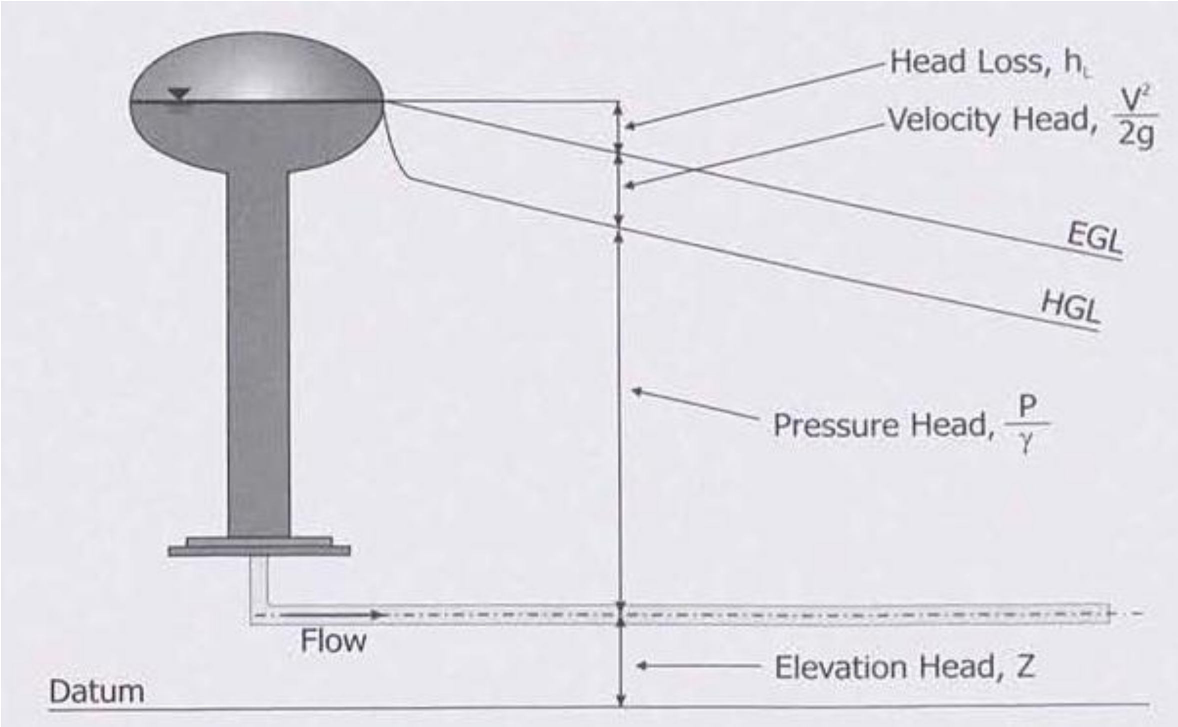
$\gamma = \rho g$ =Fluid specific weight(kg/ms)

V=velocity (m/s)

g= Acceleration due to gravity (m/s^2)

A line plotted of total head versus distance through a system is called the energy grade line (EGL).

The sum of the elevation head and pressure head yields the hydraulic grade line (HGL), which corresponds to the height that water will rise vertically in a tube attached to the pipe and open to the atmosphere [14].



Source: Thomas M., *et. al*, 2003

Figure 2.2 Energy and Hydraulic Grade Lines

In most water distribution applications, the elevation and pressure head terms are much greater than the velocity head term. For this reason, velocity head is often ignored, and modelers work in terms of hydraulic grades rather than energy grades.

Chapter 3 Methodology

3.1 Description of the Study Area

Olonlomi town is located in , Ejersa Lafa District, West Shoa Zone of Oromia Regional State of Ethiopia. It is situated in the western part of Ethiopia at a distance of 60km from Addis Ababa; on the asphalt road that leads to Ambo town. The town has about 864 hectares size.

According to the report of the Central Statistics Agency of Ethiopia (CSA, 2007) the estimated total population of the town at 2017 is about 8,059 and according to the data taken currently from the Olonkomi town municipality shows the current population of the town is 10,200.

The town is characterised by vigorous types of topography with elevation difference ranges from 2106 to 2284 meters above sea level and average temperature in between 11.54o^{c}-25.28$o^{c.}$

The water demand of the town increases due to:-

- Increasing urban population:- The population of the town increases with high flow of people from rural as the town is newly established town of Ejersa Lafo district which increases the water demand.
- Industrialization:- Different industries are going to be established in addition to the existing flower factory with full infrastructure of the town and nearness of the town to Addis Ababa and increases the water demand.
- Economic development:- The economic development of the town is increasing and the living standard of the people is departing to modern lifestyles level and cause the water consumption to rises. All these has created great burden on the water distribution system of Olonkomi town.

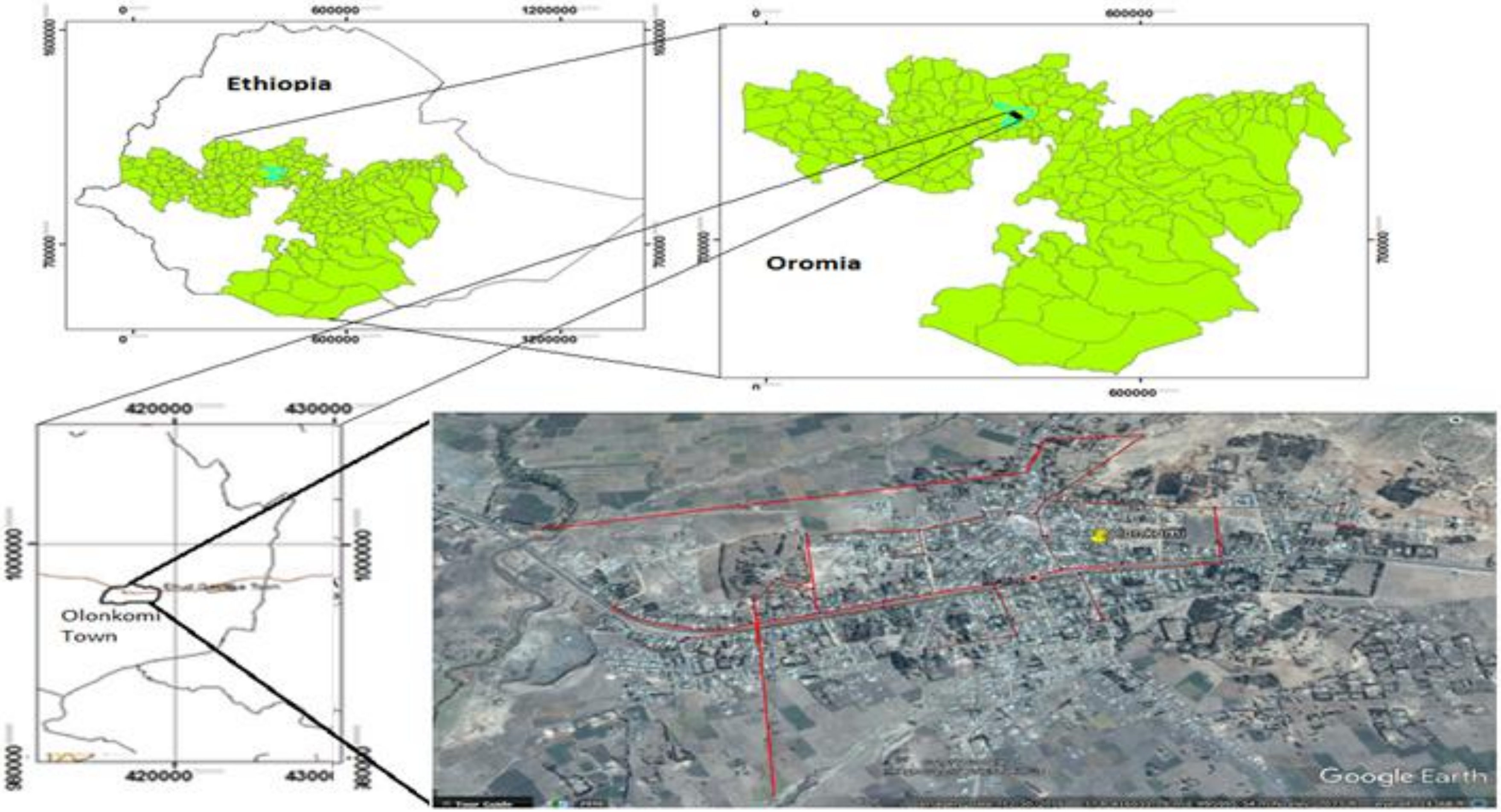


Figure 3.1 Location Map of the Study Area

3.2 Design Period

Considering factor affecting the design period, Ministry of Water, Irrigation and Electric of Ethiopia, set its own standards or planning horizons to be used for the design of water supply system. These planning horizons are [9]:

- stage one for 10 years
- stage two for 20years

This project is also designed by using the recommended planning horizons or period of water supply standard to serve the community efficiently for 20 years.

3.3 Population Projection

Population figures for the design of this project is based on the 2007 population and housing census of Ethiopia, published by the Central Statistical Agency of Ethiopia (CSA).

The Central Statistical Agency has established an annual growth rates for population projections for towns (urban) and rural areas by region.

The rate of population growth rate shows decreasing. This is due to the economic problem that forced peoples to reduce to have excess births and caused women to delay pregnancy.

For Olonkomi town because of the in-migration of people from rural areas are expected as the town was newly established town of Ejersa Lafo District, the growth rate is likely to increase. This expectation need further study of population growth rate. Hence for this design purpose, in projecting the future population sizes, the country level CSA's growth rates presented in the table below has been used.

Table 3.1 Urban Population Growth Rates

Year	Urban Growth Rate %
1995-2000	4.3
2000-2005	4.1
2005-2010	4.06
2010-2015	3.88
2015-2020	3.69
2020-2025	3.51
2025-2030	3.35

Source: Ministry of Water Resource of Ethiopia, 2006

Geometric growth method of population forecasting has been adopted for this project analysis, due to the assumption that the percentage increase in population remains constant with a constant growth rate and the country carries official surveys or censes of population at intervals of 10 years and set a constant growth rate and its applicability for growing towns.

The following formula has been adopted for the population projection [6].

$$P_n = P(1 + \frac{IG}{100})^n ------------------------Eq.3.1$$

Where

P_n = Population at the end of n future decades

P=Present population

$I_{G=}$ Average percentage increase per decade

n= Decade or year

Taking the base year population of town and the growth rate, the projected population are presented as shown on table 3.2.

Population figures projected for the base year, 2017 based on the 2007 population and housing census of Ethiopia for Olankomi town is 8, 059, but the data taken currently from the Olonkomi town municipality is 10,200. Since there is great urbanization due to the town as a capital of the district, Ejersa Lafo, newly formed district, the data taken from the municipality is used for this analysis of water demand forecast.

Table: 3.2. Projected Population of the Beneficiaries

Description	**Unit**	**Years**				
		2017	**2022**	**2027**	**2032**	**2037**
Population	Growth Rate (%)	3.69	3.51	3.35	3.19	3.03
	No	10,200	12,120	14,291	16,721	19,412

3.4 Water Demand

The rate of water, water consumption for various purposes of Olonkomi town are briefly described below:

3.4.1 Estimation of Water Demand

For the purpose of estimation of total requirements of water, the demand is calculated on an average basis expressed in litres per capita per day (lpcd) [5]. This figure represents the average daily amount of water required per person during period of a year in normal or drought conditions. It is generally estimated by dividing the annual-average daily water consumption by the total population served. The daily water consumption can be calculated by per head rate of consumption which depends on the requirements of water for various uses.

$$q = \frac{Q}{(P*365)} ----------------------------Eq.3.2$$

Where

q=Average per capita demand

Q= Total annual volume of water in litres

P= population of the town

3.4.2 Classification of Water Demands

3.4.2.1 Domestic Water Demand: is the quantity of water required for various domestic usages. This includes water requirements for drinking, cooking, bathing, washing, flushing toilets, lawn sprinkling, gardening and other household sanitation purposes in private buildings [2]. Domestic water consumption varies according to the mode of service, climatic conditions and socio-economic factors [9].

The main modes of service to be considered in the design of water supply systems are [9]:

- House connection (HC)
- Yard connection
    - Yard connection, own (YCO).
    - Yard connection, shared (YCS).
    - Public tap supplies (PT).

According to our country, Ethiopia, urban water supply design criteria, the projected per capita water demand for different mode of services are given on table 3.3.

Table: 3.3 Domestic Water Demands for Different Connection Type

Connection Type	Per Capita Water Demand l/c/day for Towns	
	Stage 1	**Stage 2**
HC	50	70
YCO	25	30
YCS	30	40
PF	20	25

Source: Ministry of Water Resource of Ethiopia, 2006

The Per capita water demand for towns indicated on table 3.3 as stage 1 and stage 2 shows that the consumption of the water demand for the first 10 years is as described in stage 1 and the consumption of water demand for the next 10 years are as described in stag 2.

In addition to the above domestic water demand given in urban water supply design criteria set in January 31, 2006, the Minister of Water , Irrigation and Electric of Ethiopia also set a revised minimum Per capita water demand (l/sec) for towns in its second growth and transformation plan (GTP-2) manual as shown on table 3.4.

Table 3.4 Domestic Water Demand for Different Population Range

Population Range	**Level of Town**	**Per Capita Water Demand l/c/day**
>1,000,000	1	100
100,000 - 1,000,000	2	80
50,000-100,000	3	60
20,000-50,000	4	50
<20,000	5	40

Source : Ministry of Water, Irrigation and Electric of Ethiopia ,2016

In GTP-2 manual, it is designed to cover 75% of the people to be user of house connection and yard connection, and 25% to be user of public tap.

But currently the data taken from the water supply and sewerage authority of the town shows that, from the user of water supply, 54% is user of house connection and yard connection and 46% is using Public tap.

Hence for this analysis purpose, the current defined percentage, 54% for house connection and yard connection and 46% for Public tap are used for the base year design and the GTP-2 design manual, 75% for house connection and yard connection and 25% for public tap are used for the design period. For per capita water demand, l/c/day, minimum value set in GTP-2 ,40 l/c/day, is used for base year design and value set in urban water supply design criteria, 70 l/c/day, is used for design period for house connection. As the same time 20 l/c/day and 25 l/c/day are used for public tap for the base and design year respectively.

3.4.2.2 Non-Domestic Water Demand

Institutional Water Demand: Institutional water demand is the quantity of water demand required for various public utility purposes of public buildings; such as city hall, castody's, schools, hospitals, Public offices etc. as well as water used for public services, including street washing, watering of parks, gardens, water fountain, swimming pools, cleaning of public sewers, etc. This quantity will certainly vary with the nature of the city and the number of institutions present in it. On an average it accounts for 5-10 present of the total domestic water demand.

Commercial Water Demand*:* Commercial water demand includes water demand for hotels, shopping centres, service stations, movie houses, airports, and the like. The commercial water demand depends on the type and number of commercial establishments. Commercial water demand is mostly about 10-20 per cent of the domestic water demand. However, the water demand in various types of commercial establishments may vary greatly.

Table 3.5 Typical Daily Water Demands of Commercial and Institution

Category	Daily Consumptions
Restaurants	10 l/seats
Boarding schools	60 l/pupil
Day school	5 l/pupil
Public offices	5 l/employee
Workshop/shops	5 l/employee
Mosques & churches	5 l/employee
Cinema house	4 l/seat
Abattoir	150 l/cow
Hospitals	50 - 75 l/bed
Hotels	25 -50 l/bed
Public bath	30 l/visitor
Railway & bus station	5 l/user
Military camps	60 l/person
Public latrines (with water facility connection)	20 litres/seat

Source: Ministry of Water Resource of Ethiopia, 2006

For the hydraulic analysis of this project 15 percent (the average) of the domestic water demand is taken for Institutional and commercial water demand.

Domestic Animal Demand

Domestic animal demand is the demand needed for livestock. In towns in the absence of any traditional water source to supplement the livestock water demand, the water demand figures given on table 3.6 is used to estimate the livestock water demand.

Table 3.6 Domestic Animal Water Demand

Livestock Type	Consumption
Cattles, donkeys, horses, etc:	50 l/head/day
Goats/sheep:	10 l/head/day
Camel	150 l/head/month

Source: Ministry of Water Resource of Ethiopia, 2006

In this analysis for the presence of rivers near the town, domestic animal demand is not considered.

Industrial Water Demand: Water required under industrial water demand depends mainly on the types of industry in the town. The water required by factories, paper mills, textile mills, breweries and sugar mills etc. comes under industrial uses. Since each industry's requirements vary, it is preferable if requirements of specific industry are worked out separately. For a city with moderate intensity of factories, the water requirements under this head may be taken 20 to 25 per cent of the per capital allowance of water [5].

Generally, large industries develop their own water supply systems. Only small industries purchase water and, therefore, imposed water on local municipal systems.

Expecting small industries in the project area, 10% of the domestic water demand is taken as industrial demand for the analysis of this town.

Fire Demand

Fire demand is the quantity of water required for fighting a fire out-break and will be particularly essential for high value of district of commercial centres, stores, etc.

Annual volumes required for firefighting purposes are generally small but during periods of need, the demand may be exceedingly large and in many cases may govern the design of distribution systems, storage, and pumping equipment.

In cost effective design water required for fire fighting shall be met by stopping supply to consumers for the required time and directing it for fire fighting purposes. Therefore, in smaller towns there is no reason to increase capacity to provide for fire fighting. However, water for fighting purposes in towns of moderate sizes is provided for as a reserve of 10% of the storage reservoir volume.

In larger towns and towns with water supply service level well above the minimum standards, economic risk analyses may need to be made to fix the level of extra cost to be incurred for fire fighting in terms of the overall cost of the water supply system.

Since the town for which this design of water supply system is considered is a smaller town there is no need to include fire fighting demand.

3.4.2.3 Unaccounted for Water (UFW)

All the water supplied into water mains, does not reach the consumer. Some portion of it is lost in pipe lines due to defective pipes joints, cracked pipes, loose valves and fittings. Some water is lost due to unauthorised and illegal connections [5].

This unaccounted system losses and leakages can be reduced by careful maintenance and universal metering.

It is well known that unaccounted for water varied according to the individual circumstances in each town.

Therefore, for the absence of past study of unaccounted for water for the specific site, unaccounted for water set as a design criteria of our country, Ethiopia are used for this specific site.

Table 3.7 Unaccounted for Water

Losses as% of Production				
Start Years	5 Years	10 Years	15 Years	20 Years
40%	35%	30%	27.5%	25%

Source: Ministry of Water Resource of Ethiopia, 2006

3.4.3 Average Day Water Demand

The average day water demand is the sum of domestic water demand, non-domestic water demand, and fire fighting water demand and unaccounted for water.

3.4.4 Demand Adjustment Factors

3.4.4.1Climatic Adjustment Factors

Climate is one of the factors that influence the quantity of water consumption and should be considered in our design. The following table shows the climatic effects factors adopted and applied to the per capita demand obtained.

Table 3.8 Climatic Effect Factors

Mean Annual Temp. (^{0}C)	Description	Altitude	Factor
<10	Cool	>3,300	0.8
10-15	Cool temperate	2,300-3,300	0.9
15-20	Temperate	1,500-2,300	1
20-25	Warm temperate	500-1,500	1.3
25 and above	Hot	<500	1.5

From the hydro-metrological data of the region, the town has a mean annual temperature of 18.41 ^{0}C with an altitude of 2195m. Therefore, a climatic adjustment factor of 1 is used to adjust the per capita average domestic water demand.

3.4.4.2 Socio-Economic Adjustment Factors

Socio-economic factors determine the degree of development of towns[9]. Therefore, the socio-economic condition of the study area plays a role in determining the water consumption of a community. Its adjustment factor is determined based on the degree of the development of the particular area under study. However, the determination of the degree of the existing development and future potential of the area depend on personal judgment.

The water supply design criteria standard set socio-economic adjustment factor for various categories/groups of development as shown on table 3.9 below.

Table 3.9 Socio-Economic Effect Factors

Group	Description	Factor
A	Towns enjoying high living standards and with high potential for development	1.1
B	Towns having a very high potential for development, but lower living standards at present	1.05
C	Towns under normal Ethiopian conditions	1
D	Advanced rural towns	0.9

Source: Ministry of Water Resource of Ethiopia, 2006

The community in the study area, as compared to other towns has a moderate socioeconomic activity, Therefore it is grouped under Group-C, Towns under normal Ethiopian conditions with socioeconomic adjustment factor of 1.

3.4.5 Maximum Seasonal Water Demand

Towns in Ethiopia are characterised by widely varying climatic conditions and so the variations in consumption during the year reflected by a peak seasonal factor will similarly vary. The seasonal peak factor adopted for any particular scheme shall be selected according to the particular climatic conditions and existing consumption records (if reliable and unsuppressed). It is expected that seasonal peak factors will vary between 1.0 and 1.2, representing the relative increase in the average daily demand during the dry and/or hot season months compared with the average annual demand

[9]. For this study the minimum value of 1 is used as maximum seasonal water demand factor.

3.4.6 Maximum Day Water Demand

Many communities exhibit a demand cycle that is higher in one day of the week than in others. This situation shall be taken into account by the use of a peak day factor. Some consultants have used peak day demand factors of between 1.0 and 1.3. The value adopted for the design of each individual scheme shall be selected according to judicious observance of the habits of consumers and the knowledge of the community and system operators. It is expected that any value selected for the peak day factor would not fall outside the above range [9]. For this study the value of 1.2 is used as maximum day water demand factor.

3.4.7 Peak Hour Water Demand

Peak hour demand represents the amount of water required during the maximum consumption hour in a given day. The distribution system must be designed to cope up with the peak demand, which is taken into account by the use of a peak hour factor. This peak hour factor is expressed as a multiple of the annual average daily demand and applied additionally to the seasonal and peak day factors. The peak hour factor varies inversely with the size of the consumer base [8].

Accordingly the following peaking factor, which correlate peaking factor with number of population, were suggested to use as design criteria, which is set by Ministry of Water, Irrigation and Electric of our country, Ethiopia.

Table 3.10 Peak Hour Factors

Population Range	**Peak Hour Factor**
< 20,000	2
20,001 to 50,000	1.9
50,001 to 100,000	1.8
>100,000	1.6

Source: Ministry of Water Resource of Ethiopia, 2006

Since the population of Olonkomi town is less than 20,000, peak hour factor of 2 is used for the analysis purpose.

3.4.8 Design Parameters of the Study

The water demand of the targeted beneficiary is studied and specified in detail so as to determine water supply components. The major types of demands assessed and considered are as summarized on table 3.11 shown below.

Table 3.11 Design Parameters of the Study

Design Parameters of the Study						
Year		**2017**	**2022**	**2027**	**2032**	**2037**
Growth Rate	Growth Rate	3.69%	3.51%	3.35%	3.19%	3.03%
Population	No	10,200	12,120	14,291	16,721	19,412
Domestic demand by Categories & Proportion of Population Served						
House Connection (54-75%) (40-70)l/c/d-	%	54	60	65	70	75
	l/c/day	40	46	53	61	70
	m^3/d	220	334	492	713	1019
Public taps users (46-25%) 20-25 l/c/d	%	46	40	35	30	25
	l/c/day	20	21	22	23	25
	m^3/d	93	101	112	117	121
Total Domestic Demand	m^3/d	314	436	604	831	1140
	L/d	314160	436329	604868	831850	1140456
Socio-economic factor	1					
Climate factor	1					
Adjusted Total Domestic demand(ATDD)	m^3/d	314	436	604	831	1140
	L/d	314160	436329	604868	831850	1140456
Non-Domestic Demand						
Public, Commercial Demand (15% ATDD)	m^3/day	47	65	90	124	171
Industrial water demand(10%ATDD)	m^3/day	31	43	60	83	114
Total Non Domestic Demands	m^3/day	78	109	151	207	285
Total Daily Demand	m^3/day	392	545	756	1039	1425

Design Parameters of the Study						
Year		**2017**	**2022**	**2027**	**2032**	**2037**
Non-Revenue Water	%	40	35	30	28	25
	m^3/day	157	190	226.83	291	356
Average Day Demand including loss	m^3/day	549	736	982.91	1330	1781
	l/s	6	8	11	15	20
Maximum Day Factor	1.2					
Maximum Day Demand	m^3/day	659	883	1179	1597	2138
	l/s	7	10	13	18	24
Peak Hour Factor	2					
Peak Hour Demand	m^3/day	1099	1472	1965	2661	3563
	l/s	12	17	22	30	41
Base flow for WP (Peak demand)	m^3/day	262	274	292	301	303
	l/s	3.04	3	3.39	3.49	3.51
Number of water point	No	13	13	13	13	13
Base flow for each WP (Peak demand)	l/s	0.23	0.24	0.26	0.25	0.27
Design pump discharge(Max day demand) considering 12hr operation	l/s	7.64	10.23	13.65	18.49	24.75
Reservoir capacity (1/3*MDD)	m^3	219	294	393	532	712

3.5. Service Reservoirs

According to Water, Irrigation and Electric Minister of our country set design criteria, the capacity of reservoir is 1/3 of maximum day demand for the absence of hourly peak factor data [8].

Table: 3.12. Reservoir Size Determination

Year	**Max. Day Demand (m^3/d)**	**Reservoir Size 1/3MDD (m^3)**
2017	659	219
2022	883	292
2027	1179	393
2032	1597	532
2037	2138	712

3.6 Transmission Main

A transmission main is the pipeline used for water transmission, that is, movement of water from the source to the treatment plant and from the plant to the distribution system [17]. The capacity of a transmission main is determined by the maximum daily water demand.

The design criteria used in the design of transmission main is flow velocities remain in the range of 0.6 to 1.5 m/s although in exceptional conditions this can rise to 2.0 m/s.

In addition the preliminary head loss (hL) in transmission main is 5.0 m/km for exceptional conditions it rise to 10.0 m/km and the allowable maximum pressure is 70 m head.

3.7 Distribution Pipe Line

The capacity of distribution pipe line is determined by the peak hour demand.

The design criteria used in the design of nodal pressure of the distribution pipe lines are as shown on table 3.13

Table 3.13 Operating Pressures in the Distribution Network

Operating Pressures	**Normal Conditions**	**Exceptional Conditions**
Minimum Maximum	15 m water head 60 m water head	10 m water head 70 m water head

Source: Ministry of Water Resource of Ethiopia, 2006

The design criteria used in the design of velocities of the distribution pipe line is 0.6 to 1.5 m/s in exceptional conditions this can rise to 2.0 m/s.

3.8 Head Losses in Pipes

Hazen-Williams formula, which is the most widely used in pressurized pipeline, that relates the velocity of the flow, hydraulic mean radius and hydraulic gradient is used in the analysis of this design to calculate head loss due to friction.

$$hL = \frac{10.7xLxQ^{1.852}}{C^{1.852}xD^{4.87}} ------------------------ Eq.3.3$$

Where,

hL = head loss due to friction (m)

L = Distance between sections or length of pipe lines (m)

C = Hazen-Williams C- Value

D = Internal diameter of pipe (m)

Q = Pipe line flow rate (m^3/s)

Table 3.14 Recommended C-Values for Various Pipe Materials.

C-Value for Hazen-Williams			
Type of Pipe	**uPVC**	**Steel**	**DCI/GI**
New	130	110	120
Existing	100-110 *	90-110 *	100-110*

Source: Ministry of Water Resource of Ethiopia, 2006

Note : - * Depending on age and condition.

3.9 Hydraulic Network Analysis

Analysis of the water supply system has been made by predefined hydraulic formula in Micro Soft Excel spreadsheet at current year daily average, at peaking hour and temporal variations of demand at design year. There are two types of analysis; steady-state analysis and extended period simulation. For this specific site since there is no hourly peak factor data and the Micro Soft Excel is unable to use extended period simulation data, only steady state analysis is practiced in Micro Soft Excel spreadsheet.

Steady state analysis is run for the demand at which every nodal demand is not changing throughout 24 hours of a day. The analysis is run for peak-hour demand condition, to check for the possible value of the minimum systems pressure and its minimum demand condition, to check for the value of the possible maximum pressure in the network.

3.10 Source of Data

The water distribution network of the existing water supply system is the main source of data for the analysis of a water distribution system and has been collected from Olonkomi town water supply and sewerage authority.

The water distribution network is available in hard copy and include the following system information.

- Pipe network alignment
- The locations of system components, such as borehole, generator house , reservoir and public taps

- Pipeline data like material type, diameter, and length.
- Elevations

In addition the production and consumption of water has been collected from Olonkomi town water supply and sewerage authority.

3.11 Input Data and Analysis

The water supply system is analysed according to the design criteria standard set in our country so as realize the systems under critical conditions.

The analysis is created using Microsoft Excel spreadsheet. To analysis the system the following input data has been entered in to the Microsoft Excel spread sheet.

- Pipe Material type
- Diameter of pipe
- Length of pipe
- Hazen-Williams Coefficients (C-Value)
- Elevation
- Demand

Demand nodes is identified based on the user community using the existing pipe line layout (with nodes) which give as a working idea of the respective number of houses within the area covered by each node.

The average day demand is the basis for the hydraulic network analysis. The demand condition has been varied by adjusting the demand factor for the average day demand condition, maximum day demand and the peak-hour demand.

The demand is distributed to all the nodes. The distribution of demands should take into consideration the relative number of houses for the different node areas.

The set-up of the Microsoft Excel work sheets table is prepared for the hydraulic analysis includes the following steps:

- Set the first column as serial number(S/N) to know visibly the sequence of each pipe member.
- Set the second column as member pipe to show the start node and the end nod of the pipes.
- Set the third column as pipe type/ material to identify the material and to use the C- value for each pipe types .
- Set the fourth column as C-value which varies with life of the pipes to use the value in the friction loss formulae.
- Set the fifth column as Year of construction of the pipe types to use the proper C-value for each year of construction.
- Set the sixth column as length in meter (m) that indicates the distance between the start node and the end nods.
- Set the seventh column as diameter of pipe (Nominal diameter) in meter (m) to distinguish the diameter of pipe length between the start node and the end nods in analysis.
- Set the eighth column as peak flow in m^3/sec to assign the water demand used at each node in the analysis.

$$Nd = P * d * Pf ------------------------- Eq.3.4$$

Where

Nd = Nodal demand

P = population of the service area

d = Per capital demand

Pf=demand factors

- Set the ninth columns as velocity of the flow in m/sec to know the velocity of flow in the pipe and compare with the design criteria standard. The known formulae to calculate the velocity of the flow is to divide the discharge by area for the known flow and area in the pipe and fix this formulae in the MS-Excel spreadsheet.

$$V = \frac{Q}{A} = 4\frac{Q}{\Pi D^2} ------------------------- Eq.3.5$$

Where

Q=Peak flow (m^3/s)

D=Pipe diameter (m) , Π=3.14

- Set the tenth column as friction loss in meter (m) to calculate the friction loss between the start node and the end nods of pipe length in the analysis. The friction loss developed by Hazen-Williams formulae indicated in equation 3.3 is used due to it is widely used in pressurized pipeline and relates the velocity of the flow, hydraulic mean radius and hydraulic gradient. The formulae is fixed in MS-Excel spreadsheet table including all the needed parameters of the formulae.
- Set the eleventh column as elevation at reservoir in meter (m) which indicates the initial point of draw out of the discharge/flow.
- Set the twelfth column as elevation at end point of each node of pipe line in meter(m) to identify the elevation at each nodes.
- Set the thirteenth column as hydrostatic head in meter (m) to designate the difference between elevation at reservoir and elevation at end points of each nodes.

$$HH = El.\mathrm{Re} - El.Ep. -------------------------- Eq.3.6$$

Where

HH=Hydrostatic Head (m)

El. Re= Elevation at Reservoir (m)

El. EP= Elevation at End point (m)

- Set the fourteenth column as cumulative friction loss .This is the total friction loss between the reservoir and the pipe nodes at concern. The friction loss between nodes are calculated using the formulae described on equation 3.3. To obtain the cumulative friction loss we have to add the value of friction loss starting from the reservoir to the point in question.

$$CFL = FLMP + FLRSN. -------------------------- Eq.3.7$$

Where

CFL=Cumulative Friction Loss (m)

FLMP=friction loss of member pipe in analysis(m)

FLRSN=friction loss from reservoir to start nodes of member pipe in analysis(m)

- Set the fifteenth column as available head in meter (m) to be aware of the net available head and compare it with the design criteria standard. Available head is the difference between hydrostatic head and cumulative friction loss.

$$AH = HH - CFL. ---------------------------- Eq.3.8$$

Where

AH=Available Head (m)

HH=Hydrostatic Head (m)

CFL= Cumulative Friction Loss (m)

After all the necessary date required by the Microsoft Excel spreadsheet is inserted, the head losses (m) in each pipe, the flow velocities (m/s), and the pressure in each node (m) are computed by the defined set-up in Microsoft Excel spreadsheet. This result obtained can help as to decide the overall design of the system as per the design criteria of the country.

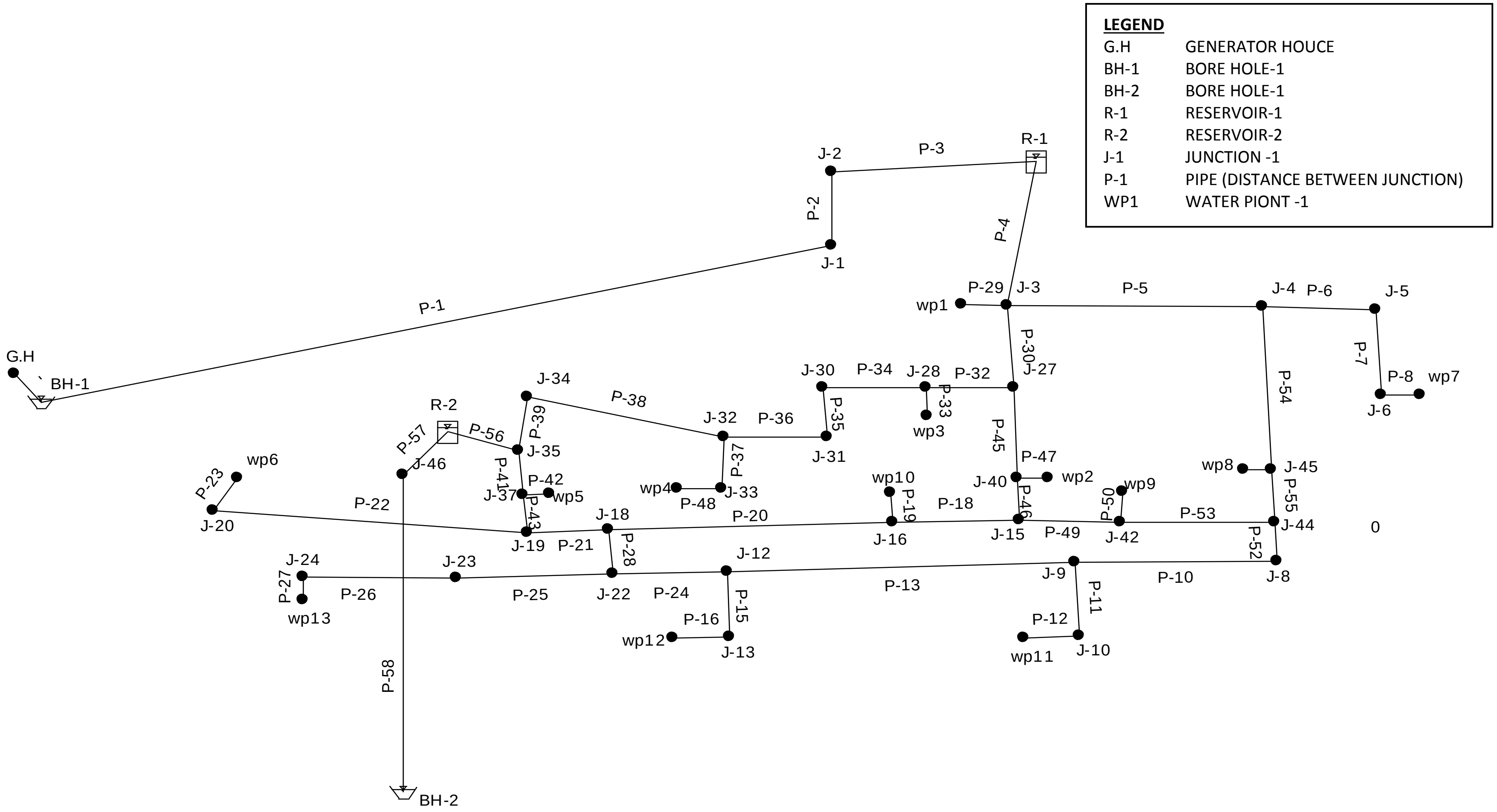
Chapter 4 Result and Discussion

4.1 Hydraulic Parameters

The possible necessary computed hydraulic parameters used to examine the project can be observed from the computer run result. The design is based on the design criteria of the water distribution system, parameters of pressure and velocity.

4.2 Existing Water Distribution Network

The existing schematized water distribution network of Olonkomi town is as shown below:



Source: West Shoa Zone Water, Mineral and Energy office, 2011

Figure 4.1 Olonkomi Town Pipe Line Water Distribution Network

Figure 4.2 Olonkomi Town Pipe Line Water Distribution Network Overlaid on Google Earth Image

4.3 Analysis Result and Discussion for Average Day Water Demand at Base Year

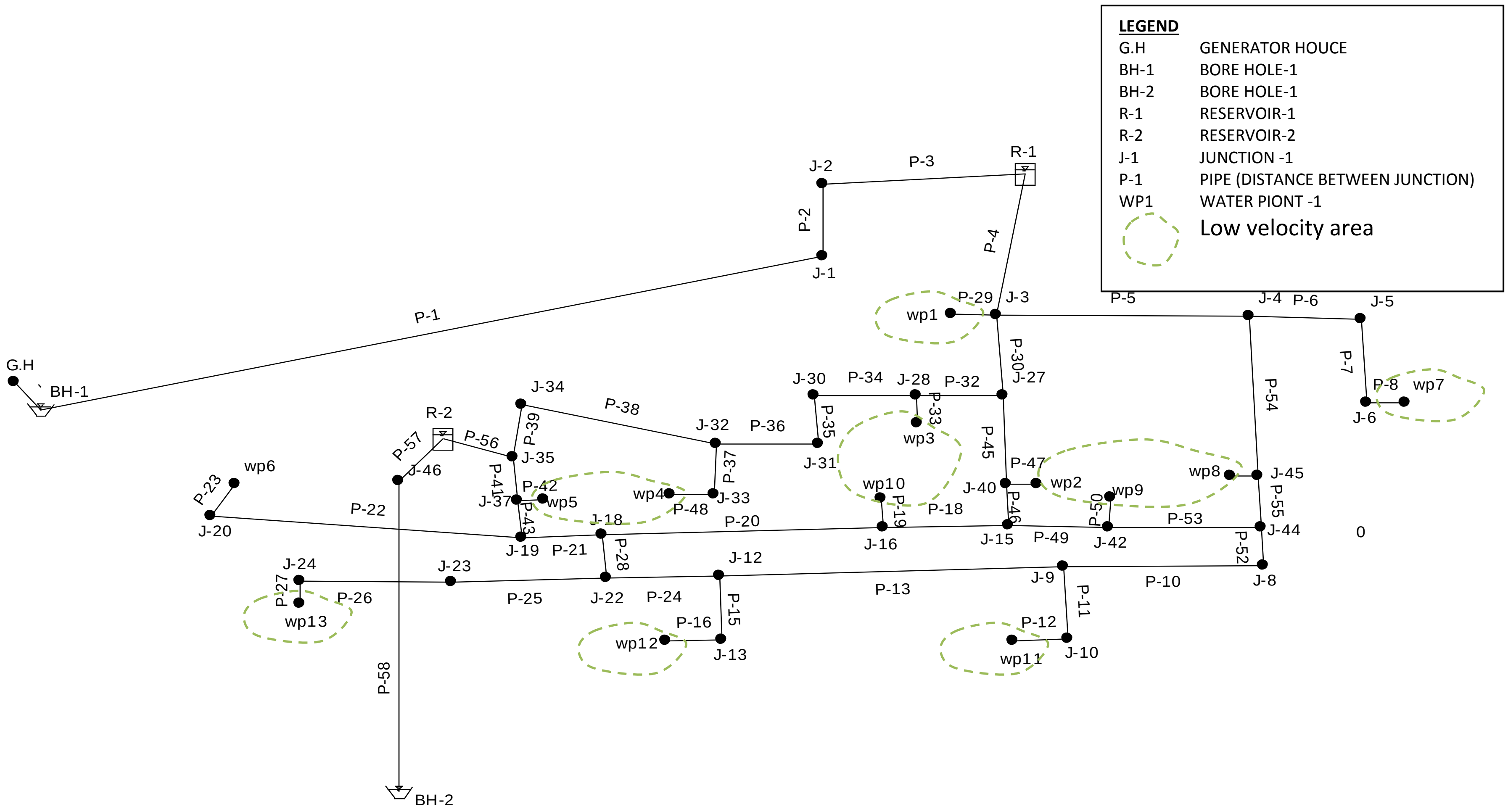
Table 4.1 Average Day Water Demand Distribution System Analysis Report at Base Year
Hydraulic Analysis Result of Olonkomi Town WSP
With Average Day Water Demand at Base Year (2017)

S/N	Member Pipe	Pipe Type	C	Year of construction	Length (m)	Dia Meter of Pipe DN (m)	Peak flow M3/se (Eq.3.4)	Velocity (m/sec (Eq.3.5)	Friction loss (m) (Eq.3.3)	Elvn. at Reservoir (m)	Elvn. at end point (m)	Hydro static head(m) (Eq.3.6)	Cumulative friction loss (m) (Eq.3.7)	Available Head(m) (Eq.3.8)
1	T1-R1	GSP	110	2005	1902	0.1	0.0076	1.0	30.3	2217.17	2137.19	80	30	
2	R1-J3	GSP	110	2005	450	0.1	0.005724	0.7	4.2	2217.17	2171.22	46	4	42
3	J3-WP1	GSP	110	2005	30	0.04	0.00012	0.1	0.0	2217.17	2171.66	46	4	41
4	J3-J27	GSP	110	2005	80	0.08	0.00280	0.6	0.6	2217.17	2170.33	47	5	42
5	J27-J28	GSP	110	2005	80	0.065	0.00252	0.8	1.3	2217.17	2171.29	46	6	40
6	J28-WP3	GSP	90	193	16	0.04	0.00012	0.1	0.0	2217.17	2170.26	47	6	41
7	J28J30	GSP	110	2005	76	0.065	0.00240	0.7	1.2	2217.17	2168.55	49	7	41
8	J30-J31	GSP	110	2005	40	0.065	0.00237	0.7	0.6	2217.17	2169.09	48	8	40
9	J31-J32	GSP	110	2005	190	0.065	0.00225	0.7	2.6	2217.17	2160.32	57	10	46
10	J32-J33	GSP	110	2005	112	0.04	0.00025	0.2	0.3	2217.17	2155.01	62	11	51
11	J33-WP4	GSP	110	2005	6	0.04	0.00017	0.1	0.0	2217.17	2155.95	61	11	50
12	J32-J34	GSP	110	2005	325	0.065	0.00215	0.6	4.0	2217.17	2153.74	63	15	49
13	J34-J35	GSP	110	2005	290	0.065	0.00194	0.6	3.0	2217.17	2156.42	61	18	43
14	J35-J37	GSP	90	1973	140	0.08	0.00184	0.4	0.7	2217.17	2146.86	70	18	52
15	J37-WP5	GSP	90	1973	6	0.04	0.00011	0.1	0.0	2217.17	2146.86	70	18	52
16	J37-J19	GSP	90	1973	26	0.08	0.00173	0.3	0.1	2217.17	2145.7	71	19	53
17	J19-J20	GSP	100	1997	620	0.025	0.00026	0.5	19.9	2217.17	2140.79	76	38	38
18	J20-WP6	GSP	90	1973	6	0.025	0.00012	0.2	0.1	2217.17	2140.79	76	38	38
19	J19-J18	GSP	90	1973	121	0.08	0.00145	0.3	0.4	2217.17	2141.76	75	19	56
20	J18-J22	GSP	100	1997	40	0.04	0.00052	0.4	0.5	2217.17	2142.78	74	19	55
21	J22-J23	GSP	100	1997	146	0.04	0.00044	0.4	1.2	2217.17	2144.9	72	21	52
22	J23-J24	GSP	100	1997	404	0.04	0.00035	0.3	2.3	2217.17	2144.03	73	23	50

23	J24-WP13	GSP	100	1997	20	0.04	0.00012	0.1	0.0	2217.17	2150.52	67	23	44
24	J18-J16	GSP	90	1973	509	0.08	0.00070	0.1	0.4	2217.17	2149.88	67	19	48
25	J16-WP10	GSP	90	1973	10	0.04	0.00012	0.1	0.0	2217.17	2150.52	67	19	48
26	J16-J15	GSP	90	1973	165	0.08	0.00047	0.1	0.1	2217.17	2151.79	65	19	46
27	J27-J40	GSP	110	2005	115	0.05	0.00051	0.3	0.4	2217.17	2163.81	53	5	48
28	J40-WP2	GSP	100	2005	10	0.04	0.00012	0.1	0.0	2217.17	2163.87	53	5	48
29	J40-J15	GSP	110	2005	165	0.05	0.00047	0.2	0.4	2217.17	2151.79	65	6	60
30	J15-J42	GSP	90	1973	154	0.065	0.00037	0.1	0.1	2217.17	2152.03	65	19	46
31	J42-WP9	GSP	90	1973	10	0.04	0.00012	0.1	0.0	2217.17	2152.19	65	19	46
32	J42-J44	GSP	90	1973	284	0.065	0.00013	0.0	0.0	2217.17	2152.2	65	19	46
33	J44-WP8	GSP	90	1973	10	0.04	0.00012	0.1	0.0	2217.17	2152.71	64	19	45
34	J3-J4	GSP	110	2005	500	0.065	0.00200	0.6	5.4	2217.17	2162.88	54	10	45
35	J4-J5	GSP	110	2005	310	0.05	0.00040	0.2	0.6	2217.17	2167.96	49	10	39
36	J5-J6	GSP	110	2005	127	0.04	0.00032	0.3	0.5	2217.17	2161.57	56	11	45
37	J6-WP7	GSP	110	2005	10	0.04	0.00012	0.1	0.0	2217.17	2162.29	55	11	44
38	J4-J45	GSP	110	2005	230	0.065	0.00160	0.5	1.7	2217.17	2152.58	65	11	53
39	J45-J8	GSP	110	2005	46	0.065	0.00128	0.4	0.2	2217.17	2151.13	66	12	55
40	J8-J9	GSP	110	2005	400	0.065	0.00032	0.1	0.1	2217.17	2151.44	66	12	54
41	J9-J10	GSP	110	2005	210	0.05	0.00026	0.1	0.2	2217.17	2141.62	76	12	64
42	J10-WP11	GSP	110	2005	40	0.04	0.00012	0.1	0.0	2217.17	2140.61	77	12	65
43	J9-J12	GSP	110	2005	320	0.05	0.00077	0.4	2.1	2217.17	2147.14	70	14	56
44	J12-J13	GSP	110	2005	210	0.05	0.00062	0.3	0.9	2217.17	2138.52	79	15	64
45	J13-WP12	GSP	110	2005	116	0.04	0.00012	0.1	0.1	2217.17	2136.43	81	15	66

LEGEND

G.H Generator House
R-1 Bore Hole
T-1 Reservoir
J-1 Junction One
P-1 Pipe (Distance Between Junction)
Wp1 Water Point One
C Hazen-Williams C- Value
GSP Galvanized Steel Pipe



Source: West Shoa Zone Water, Mineral and Energy office, 2011

Figure 4.3 Olonkomi Town Pipe Line Water Distribution Network Analysis for Average Day Water Demand at Base Year

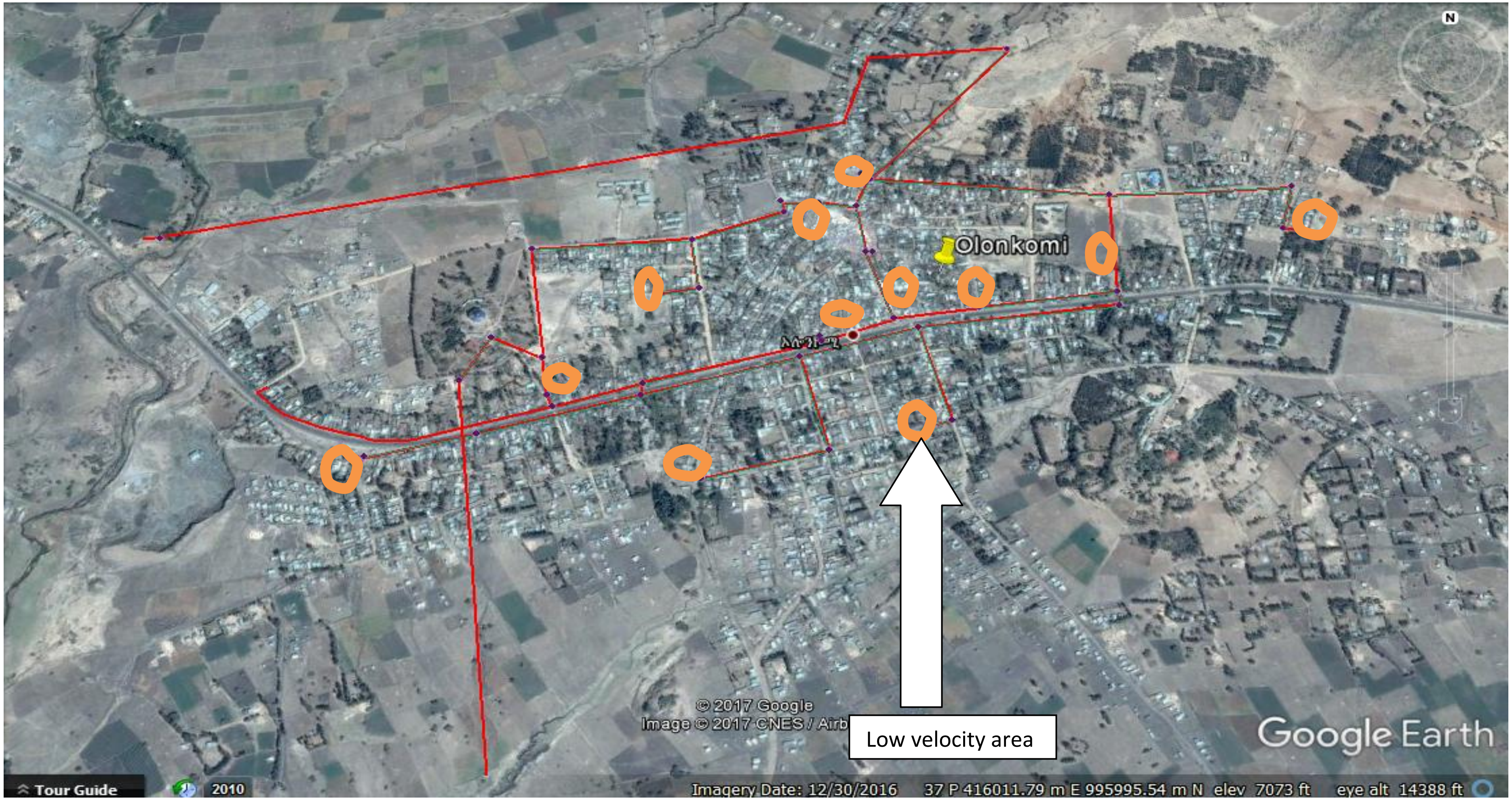


Figure 4.4 Olonkomi Town Pipe Line Water Distribution Network Analysis Overlaid on Google Earth Image for Average Day Water Demand at Base Year

As we observe from table 4.1, the hydraulic analysis result of Olonkomi town with the average day water demand at the base year indicates that the minimum available heads is 37m and the maximum available head is 65m.

Hence when we compare the hydraulic analysis result with the operating pressure in the distribution network, the resulting pressures at all the junctions are adequate enough to provide water to the user community.

However, the velocity of flow is below the standard at public taps which need the reduction of pipe size to increase the velocity of flow in the pipe. As standard the acceptable pressures at public taps should be limited to a range of 2 to 5 metres using a suitable pressure reducing valve [3].

As can be seen from Fig 4.3 & 4.4 above, the location of low velocity area is clearly indicated with a velocity of less than 0.6 L/sec.

The following figure is a plot that showed the variation of available head between nodes on the water distribution network with respect to friction loss.

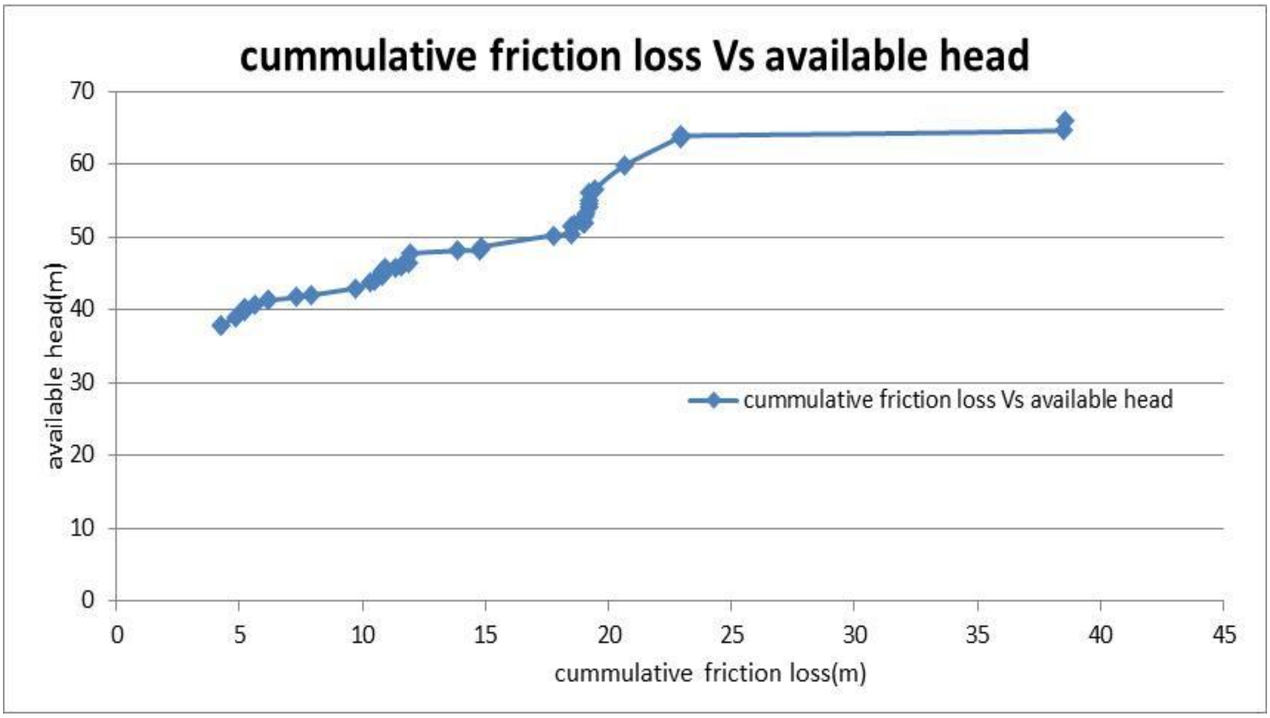


Figure 4.5 Available Head Variation at Junction Average Day Water Demand at Base Year

A plot that showed the variation of velocity of flow between nodes on the water distribution network indicates that there is a drop in the velocities then the recommended value. The recommended velocities should be between 0.6 and 2.0 m/s. The low velocities are undesirable because they lead to low pipe flows and also undesirable for reasons of hygiene.

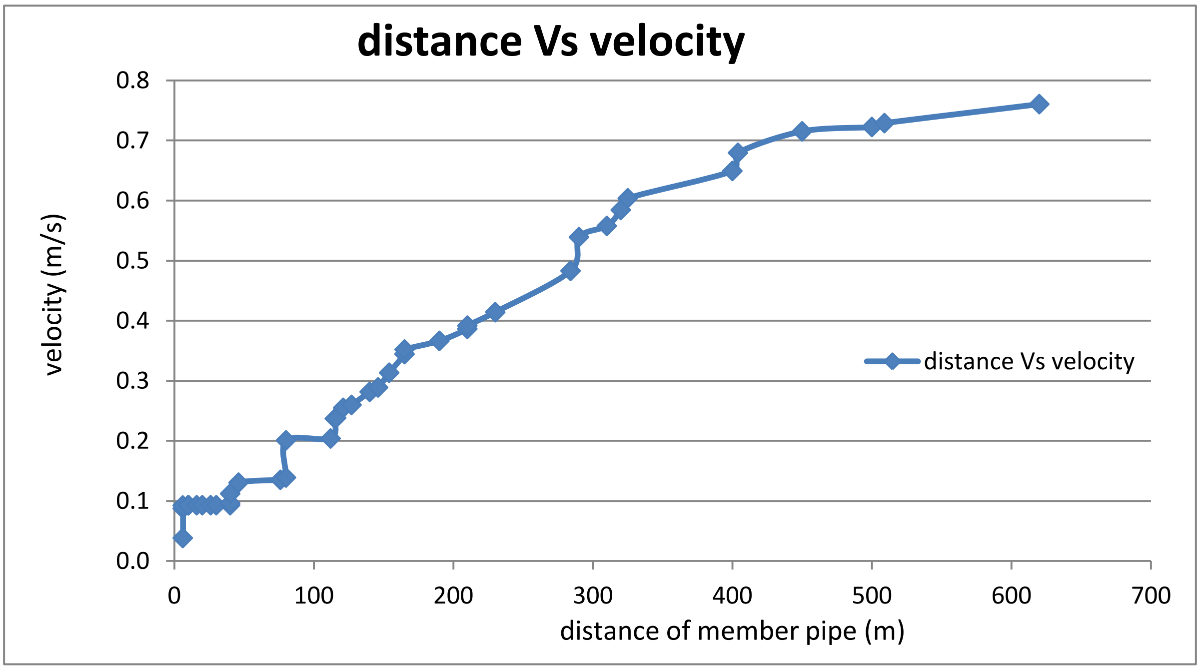


Figure 4.6 Velocity Variations in Pipe Line Network for Average Day Water Demand at Base Year

4.4 Analysis Result and Discussion for Peak Hour Day Water Demand at Base Year

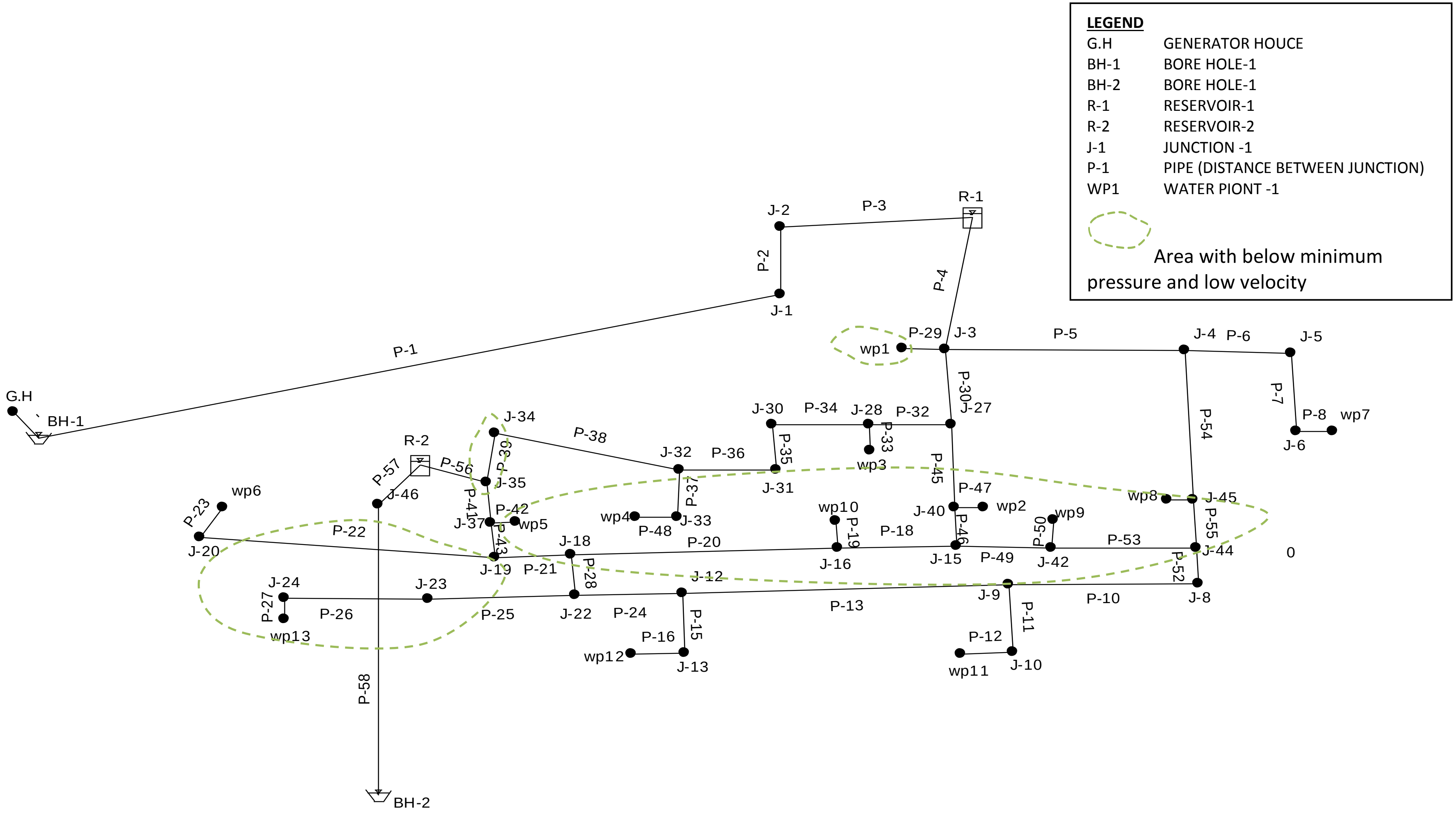
Table 4.2 Peak Hour Day Water Demand Distribution System Analysis Report at Base Year
Hydraulic Analysis Result of Olonkomi Town WSP
With Peak Flow at Base Year (2017)

S/N	Member Pipe	Pipe Type	C	Year of construction	Length (m)	Dia Meter of Pipe DN (m)	Peak flow M3/se (Eq.3.4)	Velocity (m/sec (Eq.3.5)	Friction loss (m) (Eq.3.3)	Elvn. at Reservoir (m)	Elvn. at end point (m)	Hydro static head(m) (Eq.3.6)	Cumulative friction loss (m) (Eq.3.7)	Available Head(m) (Eq.3.8)
1	T1-R1	GSP	110	2005	1902	0.1	0.0076	1.0	30.3	2217.17	2137.19	80	30	
2	R1-J3	GSP	110	2005	450	0.1	0.011457	1.5	15.2	2217.17	2171.22	46	15	31
3	J3-WP1	GSP	110	2005	30	0.04	0.00023	0.2	0.1	2217.17	2171.66	46	15	30
4	J3-J27	GSP	110	2005	80	0.08	0.00561	1.1	2.1	2217.17	2170.33	47	17	30
5	J27-J28	GSP	110	2005	80	0.065	0.00505	1.5	4.8	2217.17	2171.29	46	22	24
6	J28-WP3	GSP	90	1973	16	0.04	0.00023	0.2	0.0	2217.17	2170.26	47	22	25
7	J28J30	GSP	110	2005	76	0.065	0.00480	1.4	4.2	2217.17	2168.55	49	26	22
8	J30-J31	GSP	110	2005	40	0.065	0.00475	1.4	2.2	2217.17	2169.09	48	28	20
9	J31-J32	GSP	110	2005	190	0.065	0.00451	1.4	9.3	2217.17	2160.32	57	38	19
10	J32-J33	GSP	110	2005	112	0.04	0.00028	0.2	0.3	2217.17	2155.01	62	38	24
11	J33-WP4	GSP	110	2005	6	0.04	0.00023	0.2	0.0	2217.17	2155.95	61	38	23
12	J32-J34	GSP	110	2005	325	0.065	0.00430	1.3	14.6	2217.17	2153.74	63	53	11
13	J34-J35	GSP	110	2005	290	0.065	0.00387	1.2	10.7	2217.17	2156.42	61	63	-3
14	J35-J37	GSP	90	1973	140	0.08	0.00368	0.7	2.5	2217.17	2146.86	70	66	4
15	J37-WP5	GSP	90	1973	6	0.04	0.00023	0.2	0.0	2217.17	2146.86	70	66	4
16	J37-J19	GSP	90	1973	26	0.08	0.00345	0.7	0.4	2217.17	2145.7	71	66	5
17	J19-J20	GSP	100	1997	620	0.025	0.00030	0.6	24.9	2217.17	2140.79	76	91	-15
18	J20-WP6	GSP	90	1973	6	0.025	0.00023	0.5	0.2	2217.17	2140.79	76	91	-15
19	J19-J18	GSP	90	1973	121	0.08	0.00312	0.6	1.6	2217.17	2141.76	75	68	8
20	J18-J22	GSP	100	1997	40	0.04	0.00085	0.7	1.1	2217.17	2142.78	74	69	5
21	J22-J23	GSP	100	1997	146	0.04	0.00073	0.6	3.1	2217.17	2144.9	72	72	0
22	J23-J24	GSP	100	1997	404	0.04	0.00058	0.5	5.6	2217.17	2144.03	73	78	-5

23	J24-WP13	GSP	100	1997	20	0.04	0.00023	0.2	0.1	2217.17	2150.52	67	78	-11
24	J18-J16	GSP	90	1973	509	0.08	0.00170	0.3	2.2	2217.17	2149.88	67	68	-1
25	J16-WP10	GSP	90	1973	10	0.04	0.00023	0.2	0.0	2217.17	2150.52	67	68	-2
26	J16-J15	GSP	90	1973	165	0.08	0.00117	0.2	0.4	2217.17	2151.79	65	69	-3
27	J27-J40	GSP	110	2005	115	0.05	0.00079	0.4	0.8	2217.17	2163.81	53	18	35
28	J40-WP2	GSP	100	2005	10	0.04	0.00023	0.2	0.0	2217.17	2163.87	53	18	35
29	J40-J15	GSP	110	2005	165	0.05	0.00117	0.6	2.4	2217.17	2151.79	65	21	45
30	J15-J42	GSP	90	1973	154	0.065	0.00094	0.3	0.6	2217.17	2152.03	65	69	-4
31	J42-WP9	GSP	90	1973	10	0.04	0.00023	0.2	0.0	2217.17	2152.19	65	69	-4
32	J42-J44	GSP	90	1973	284	0.065	0.00035	0.1	0.2	2217.17	2152.2	65	70	-5
33	J44-WP8	GSP	90	1973	10	0.04	0.00023	0.2	0.0	2217.17	2152.71	64	69	-5
34	J3-J4	GSP	110	2005	500	0.065	0.00401	1.2	19.7	2217.17	2162.88	54	35	19
35	J4-J5	GSP	110	2005	310	0.05	0.00080	0.4	2.2	2217.17	2167.96	49	37	12
36	J5-J6	GSP	110	2005	127	0.04	0.00064	0.5	1.8	2217.17	2161.57	56	39	17
37	J6-WP7	GSP	110	2005	10	0.04	0.00023	0.2	0.0	2217.17	2162.29	55	39	16
38	J4-J45	GSP	110	2005	230	0.065	0.00321	1.0	6.0	2217.17	2152.58	65	41	24
39	J45-J8	GSP	110	2005	46	0.065	0.00257	0.8	0.8	2217.17	2151.13	66	42	24
40	J8-J9	GSP	110	2005	400	0.065	0.00064	0.2	0.5	2217.17	2151.44	66	42	24
41	J9-J10	GSP	110	2005	210	0.05	0.00051	0.3	0.7	2217.17	2141.62	76	43	33
42	J10-WP11	GSP	110	2005	40	0.04	0.00023	0.2	0.1	2217.17	2140.61	77	43	34
43	J9-J12	GSP	110	2005	320	0.05	0.00154	0.8	7.7	2217.17	2147.14	70	50	20
44	J12-J13	GSP	110	2005	210	0.05	0.00123	0.6	3.3	2217.17	2138.52	79	53	25
45	J13-WP12	GSP	110	2005	116	0.04	0.00023	0.2	0.2	2217.17	2136.43	81	53	27

LEGEND

G.H Generator House
R-1 Bore Hole
T-1 Reservoir
J-1 Junction One
P-1 Pipe (Distance Between Junction)
Wp1 Water Point One
C Hazen-Williams C- Value
GSP Galvanized Steel Pipe



Source: West Shoa Zone Water, Mineral and Energy office, 2011

Figure 4.7 Olonkomi Town Pipe Line Water Distribution Network Analysis for Peak Hour Day Water Demand at Base Year

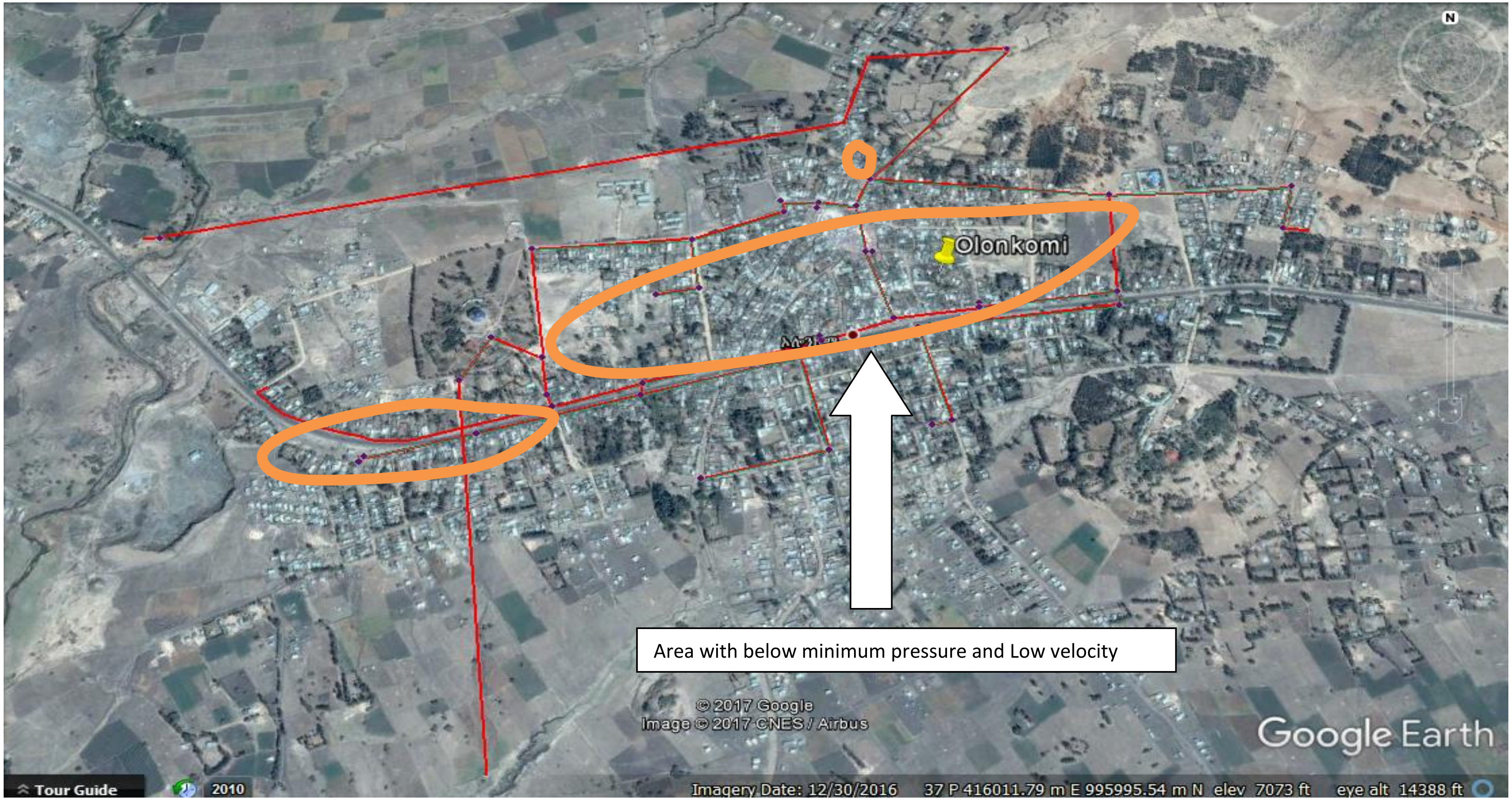


Figure 4.8 Olonkomi Town Pipe Line Water Distribution Network Analysis Overlaid on Google Earth Image for Peak Hour Day Water Demand at Base Year

The result of the hydraulic analysis of the water distribution network with the peak hour flow at the base year described in the above table 4.2 indicates that the available head at various water distribution network junctions for the network of Olonkomi town are in the range of negative pressure of 14m and maximum pressure of 30m.

Accordingly in the analysis of water supply network carried out that there are some points having water pressure less than accepted limits that are below the minimum pressure recommended under the design criteria, which fixes the minimum operating pressure in the distribution network to be 15m for normal condition and goes to 10m for exceptional conditions.

Those points below acceptable minimum pressure when analysis is done for peak demand at base year are pipe line J35-J37, J37-WP5, J37-J19, J19-J18, J18-J22 and J22-J23. As information is taken from Olonkomi town water supply and sewerage authority during field survey, these points does not gate water during peak period. Hence these pipe lines need modification according to the design criteria. In addition, the construction years of the pipe is also above its service life which is above 30 years set as standard criteria for steel pipe and has to be replaced with new.

Negative pressure is also observed while the analysis is done for the peak demand at the base year which means water does not reach the junctions completely. These points are J34-J35, J19-J20, J23-J24, J24-WP13, J18-J16, J16-WP10, J16-J15, J15-J42, J42-WP9, J42-J44 and J44-WP8. Therefore these pipe lines need modification according to the design criteria. In addition, the construction years of the pipe is also above its service life which is above 30 years set as standard criteria for steel pipe and has to be replaced with new.

In addition to the minimum pressure result of the analysis output, there are some pipes with low velocity than the recommended value set in the design criteria. The design criteria set the highest velocity to be 2 m/sec and the minimum velocity to be 0.6m/sec. Those points are at: J3-WP1, J28-WP3, J32-J33, J33-WP4, J37-WP5, J20-WP6, J23-J24 and J24-WP13. Hence these pipes should be adjusted in between the design criteria set in order to avoid stagnation and water quality problems in the water systems.

In addition Figure 4.7 & 4.8 above shows clearly Area with minimum pressure below 10m and Low velocity below 0.6 L/Sec.

The following figure 4.9 is a plot that showed the variation of available head between a node on the water distribution network with respect to friction loss which indicates the increase of cumulative friction loss with increase in available head.

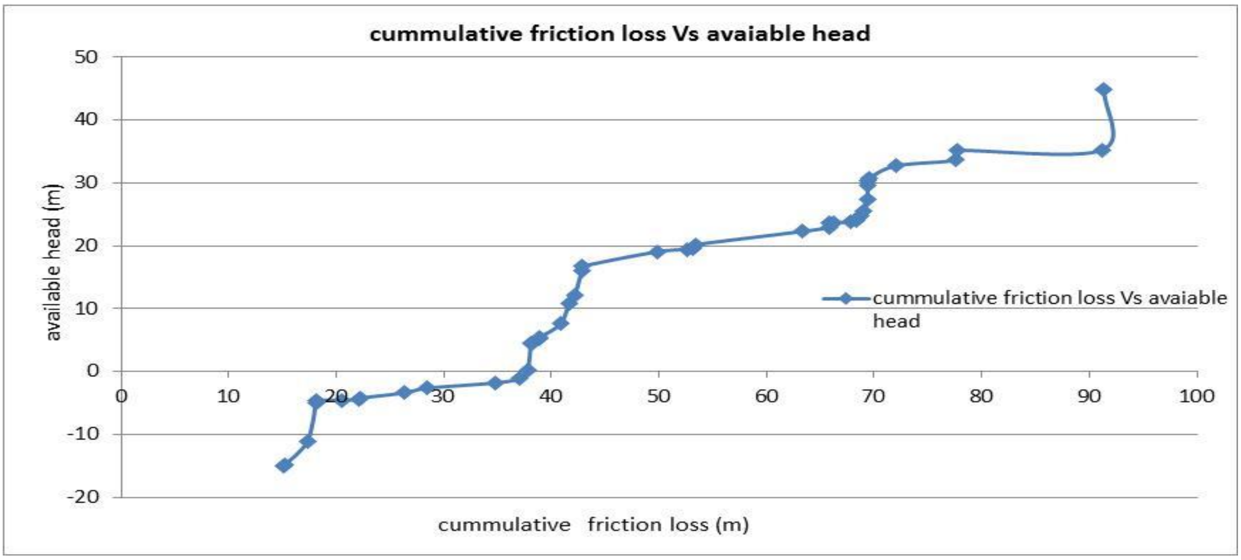


Figure 4.9 Available Head Variation at Junction for Peak Hour Day Demand at Base Year

The following plot shows the variation of velocity of flow between a node on the water distribution network which indicates that there is a drop in the velocities then the recommended value. The recommended velocities should be between 0.6 and 2.0 m/s. The low velocities are undesirable because they lead to low pipe flows and also undesirable for reasons of hygiene.

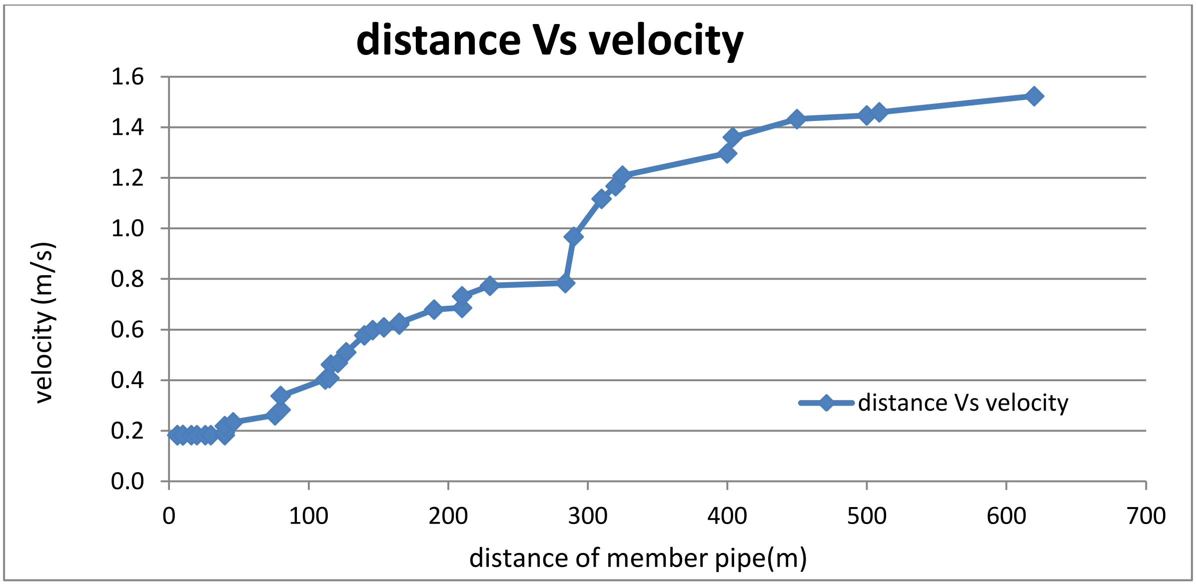


Figure 4.10 Velocity Variations in Pipe Line Network for Peak Hour Day Water Demand at Base Year

4.5 Analysis Result and Discussion for Average Day Water Demand at Design Year

Table 4.3 Average Day Water Demand Distribution System Analysis Report at Design Year

Hydraulic Analysis Result of Olonkomi Town WSP

With Average Demand at Design Year (2037)

S/ N	Member Pipe	Pipe Type	C	Year of construction	Length (m)	Dia Meter of Pipe DN (m)	Peak flow M3/se (Eq.3.4)	Velocity (m/sec (Eq.3.5)	Friction loss (m) (Eq.3.3)	Elvn. at Reservoir (m)	Elvn. at end point (m)	Hydro static head(m) (Eq.3.6)	Cumulative friction loss (m) (Eq.3.7)	Available Head(m) (Eq.3.8)
1	T1-R1	GSP	95	2005	1902	0.1	0.0076	1.0	39.7	2217.17	2137.19	80	40	
2	R1-J3	GSP	95	2005	450	0.1	0.018540	2.4	48.6	2217.17	2171.22	46	49	-3
3	J3-WP1	GSP	95	2005	30	0.04	0.00011	0.1	0.0	2217.17	2171.66	46	49	-3
4	J3-J27	GSP	95	2005	80	0.08	0.00922	1.8	7.0	2217.17	2170.33	47	56	-9
5	J27-J28	GSP	95	2005	80	0.065	0.00829	2.5	15.9	2217.17	2171.29	46	71	-26
6	J28-WP3	GSP	90	193	16	0.04	0.00011	0.1	0.0	2217.17	2170.26	47	71	-25
7	J28J30	GSP	95	2005	76	0.065	0.00788	2.4	13.7	2217.17	2168.55	49	85	-37
8	J30-J31	GSP	95	2005	40	0.065	0.00780	2.4	7.1	2217.17	2169.09	48	92	-44
9	J31-J32	GSP	95	2005	190	0.065	0.00741	2.2	30.6	2217.17	2160.32	57	123	-66
10	J32-J33	GSP	95	2005	112	0.04	0.00030	0.2	0.5	2217.17	2155.01	62	123	-61
11	J33-WP4	GSP	95	2005	6	0.04	0.00011	0.1	0.0	2217.17	2155.95	61	123	-62
12	J32-J34	GSP	95	2005	325	0.065	0.00705	2.1	47.8	2217.17	2153.74	63	171	-108
13	J34-J35	GSP	90	2005	290	0.065	0.00635	1.9	38.7	2217.17	2156.42	61	210	-149
14	J35-J37	GSP	90	1973	140	0.08	0.00603	1.2	6.2	2217.17	2146.86	70	216	-146
15	J37-WP5	GSP	90	1973	6	0.04	0.00011	0.1	0.0	2217.17	2146.86	70	216	-146
16	J37-J19	GSP	90	1973	26	0.08	0.00592	1.2	1.1	2217.17	2145.7	71	217	-146
17	J19-J20	GSP	90	1997	620	0.025	0.00035	0.7	40.2	2217.17	2140.79	76	257	-181
18	J20-WP6	GSP	90	1973	6	0.025	0.00011	0.2	0.0	2217.17	2140.79	76	257	-181
19	J19-J18	GSP	90	1973	121	0.08	0.00552	1.1	4.5	2217.17	2141.76	75	222	-146
20	J18-J22	GSP	90	1997	40	0.04	0.00133	1.1	3.2	2217.17	2142.78	74	225	-150
21	J22-J23	GSP	90	1997	146	0.04	0.00113	0.9	8.5	2217.17	2144.9	72	233	-161
22	J23-J24	GSP	90	1997	404	0.04	0.00091	0.7	15.6	2217.17	2144.03	73	249	-176
23	J24-WP13	GSP	90	1997	20	0.04	0.00011	0.1	0.0	2217.17	2150.52	67	249	-182

24	J18-J16	GSP	90	1973	509	0.08	0.00314	0.6	6.7	2217.17	2149.88	67	224	-157
25	J16-WP10	GSP	90	1973	10	0.04	0.00011	0.1	0.0	2217.17	2150.52	67	224	-157
26	J16-J15	GSP	90	1973	165	0.08	0.00242	0.5	1.3	2217.17	2151.79	65	225	-160
27	J27-J40	GSP	95	2005	115	0.05	0.00115	0.6	2.1	2217.17	2163.81	53	58	-4
28	J40-WP2	GSP	95	2005	10	0.04	0.00011	0.1	0.0	2217.17	2163.87	53	58	-4
29	J40-J15	GSP	95	2005	165	0.05	0.00242	1.2	12.0	2217.17	2151.79	65	70	-4
30	J15-J42	GSP	90	1973	154	0.065	0.00194	0.6	2.3	2217.17	2152.03	65	227	-162
31	J42-WP9	GSP	90	1973	10	0.04	0.00011	0.1	0.0	2217.17	2152.19	65	227	-162
32	J42-J44	GSP	90	1973	284	0.065	0.00091	0.3	1.0	2217.17	2152.2	65	228	-164
33	J44-WP8	GSP	90	1973	10	0.04	0.00011	0.1	0.0	2217.17	2152.71	64	227	-163
34	J3-J4	GSP	95	2005	500	0.065	0.00649	2.0	62.9	2217.17	2162.88	54	111	-57
35	J4-J5	GSP	95	2005	310	0.05	0.00130	0.7	7.1	2217.17	2167.96	49	119	-69
36	J5-J6	GSP	95	2005	127	0.04	0.00104	0.8	5.7	2217.17	2161.57	56	124	-69
37	J6-WP7	GSP	95	2005	10	0.04	0.00011	0.1	0.0	2217.17	2162.29	55	124	-69
38	J4-J45	GSP	95	2005	230	0.065	0.00519	1.6	19.1	2217.17	2152.58	65	131	-66
39	J45-J8	GSP	95	2005	46	0.065	0.00415	1.3	2.5	2217.17	2151.13	66	133	-67
40	J8-J9	GSP	95	2005	400	0.065	0.00104	0.3	1.7	2217.17	2151.44	66	135	-69
41	J9-J10	GSP	95	2005	210	0.05	0.00083	0.4	2.1	2217.17	2141.62	76	137	-61
42	J10-WP11	GSP	95	2005	40	0.04	0.00011	0.1	0.0	2217.17	2140.61	77	137	-60
43	J9-J12	GSP	95	2005	320	0.05	0.00249	1.3	24.6	2217.17	2147.14	70	159	-89
44	J12-J13	GSP	95	2005	210	0.05	0.00199	1.0	10.7	2217.17	2138.52	79	170	-91
45	J13-WP12	GSP	95	2005	116	0.04	0.00011	0.1	0.1	2217.17	2136.43	81	170	-89

LEGEND

G.H Generator House
R-1 Bore Hole
T-1 Reservoir
J-1 Junction One
P-1 Pipe (Distance Between Junction)
Wp1 Water Point One
C Hazen-Williams C- Value
GSP Galvanized Steel Pipe

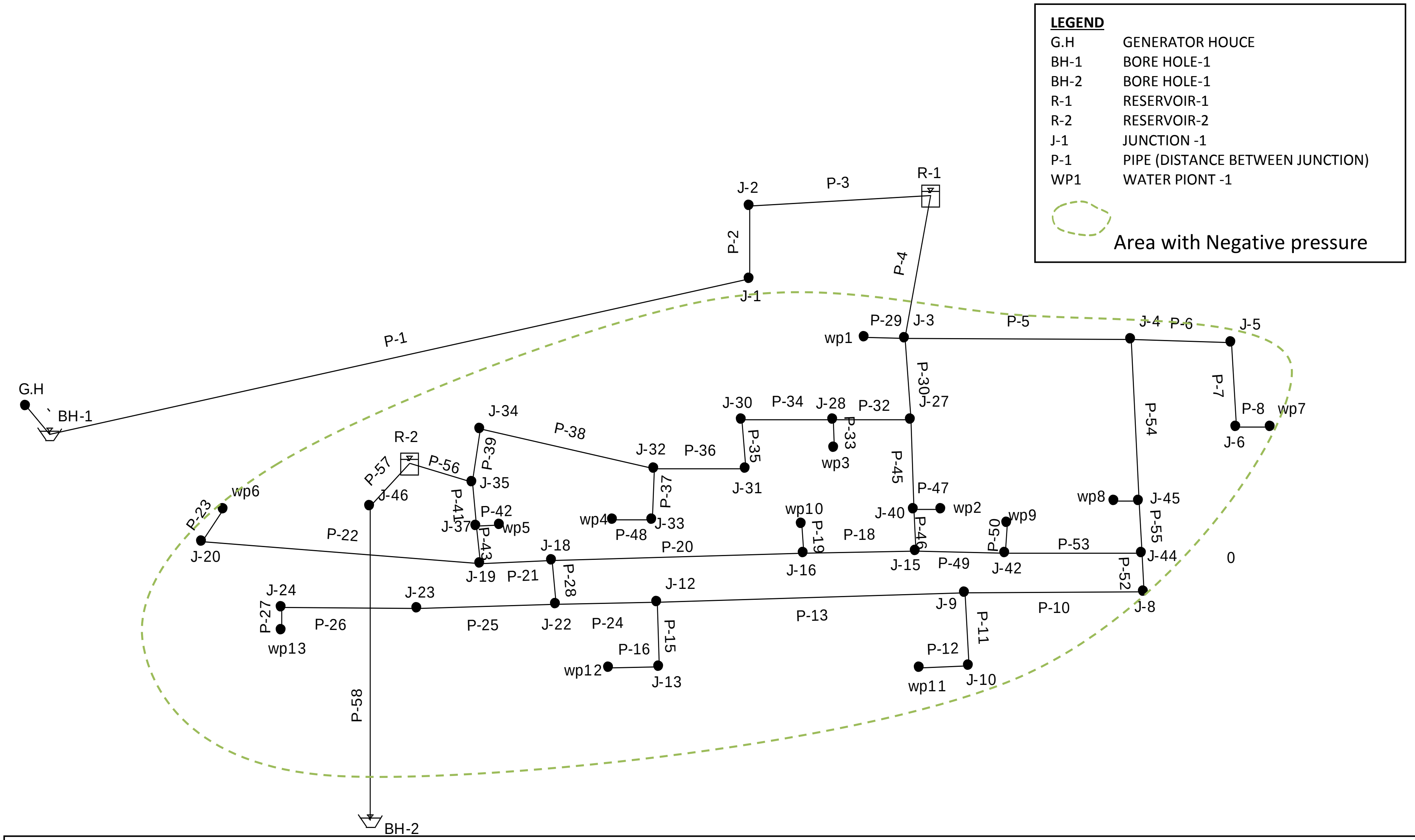


Figure 4.11 Olonkomi Town Pipe Line Water Distribution Network Analysis for Average Day Water Demand at Design Year

Source: West Shoa Zone Water, Mineral and Energy office, 2011

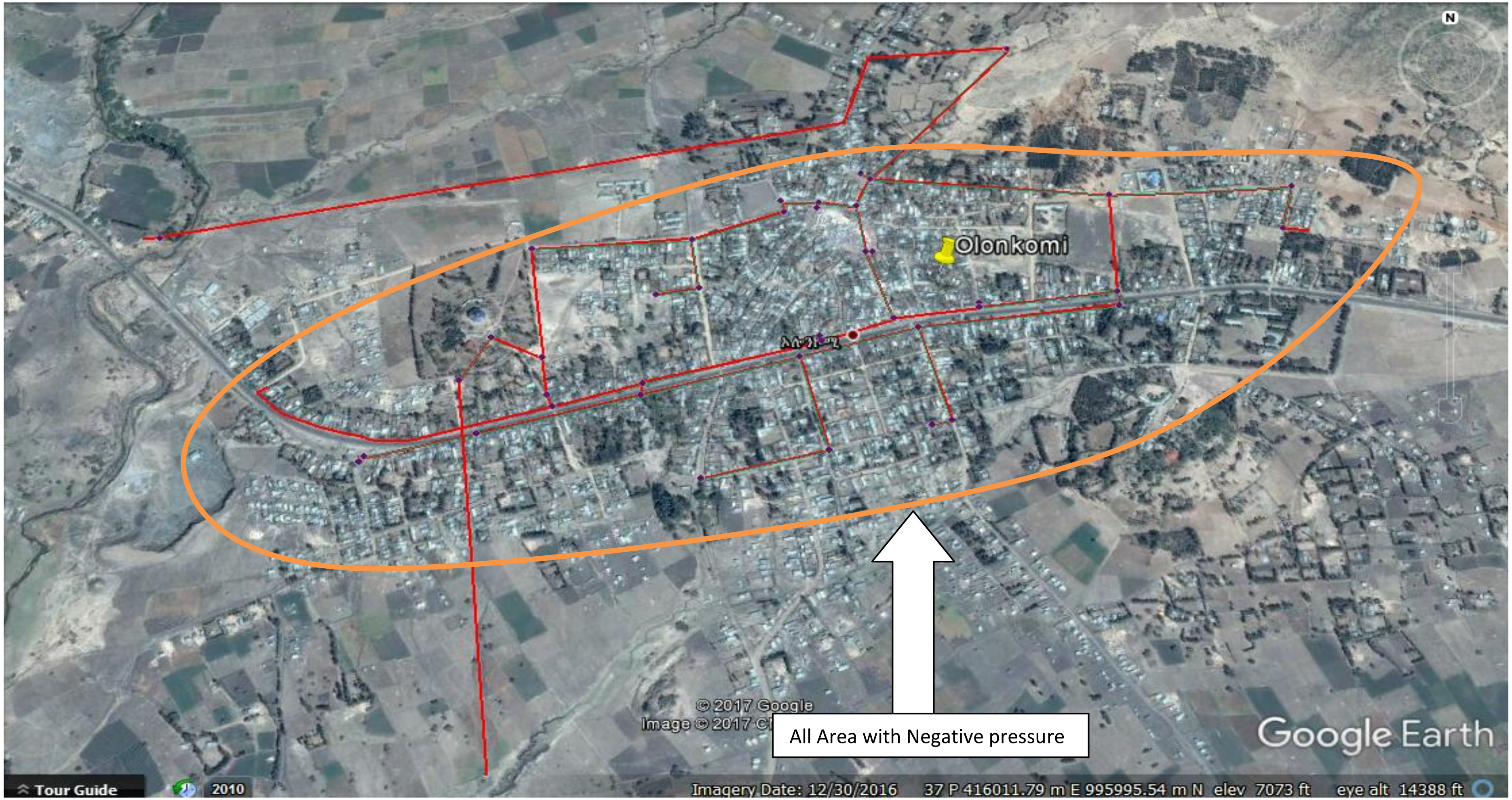


Figure 4.12 Olonkomi Town Pipe Line Water Distribution Network Analysis Overlaid on Google Earth Image for Average Day Water Demand at Design Year

As we observe from table 4.3 above, the hydraulic analysis result of Olonkomi town with the average day water demand at the design year(2037) indicate that all the junctions at all pipes shows negative pressure which means water is not completely reach the community. In addition, the velocity of flow is also not within the design criteria standard set. Therefore, complete replacement with full design is needed at the design year.

Figure 4.11 & 4.12 above shows clearly area with negative pressure indicating that water is not reach the points at all.

The following figure is a plot that showed the variation of available head between nodes on the water distribution network with respect to friction loss. As shown in the graph the available head is negative at all junctions.

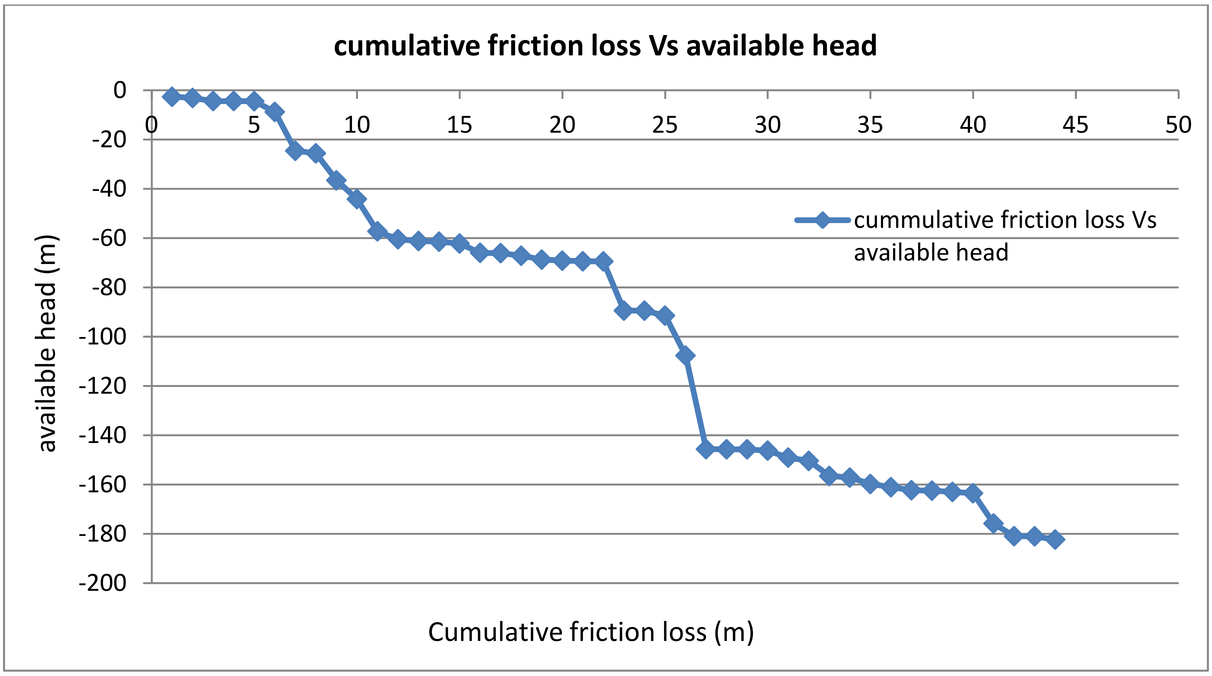


Figure 4.13 Available Head Variation at Junction for Average Day Demand at Design Year

The following plot shows the variation of velocity of flow between a node on the water distribution network which indicates that there is a drop in the velocities then the recommended value and over velocity above the recommended value. Generally, it needs redesign of all the systems at the design year.

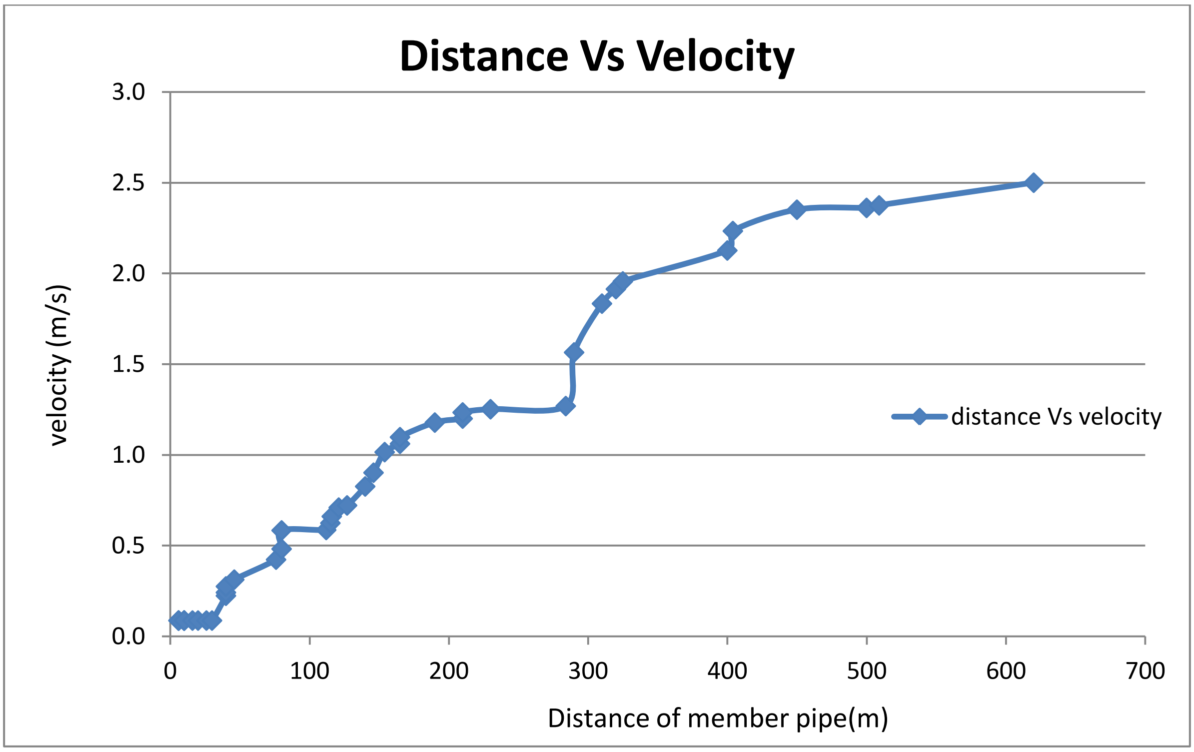


Figure 4.4 Velocity Variations in Pipe Line Network for Average Day Demand at Design Year

Chapter 5 Conclusion and Recommendation

5.1 Conclusion

Considering the design criteria, the results of the hydraulic analysis shows that acceptable minimum pressure has not been met, some of the distribution system get water with low pressure and some of them even does not gate water at all due to the pressure in the distribution system is below the permissible minimum requirement.

Accordingly from the total pipe line about 44% of the pipes need replacement even currently as shown in the peak hour demand distribution analysis report at the base year and completely all pipes replacement at the design year by resizing under and oversized distribution pipes to the provide adequate water supply for the needy community within the acceptable minimum and maximum pressure..

5.2 Recommendations

- The smaller diameter pipes that need replacement as indicated in the peak hour demand distribution analysis report at the base year and design year should have to be replaced with new one to solve the water distribution system pressure problem and deliver sufficient quantity of water supply within acceptable pressure for the needy community .
- Since the production of water per day from the wells are less than the demand needed additional sources are needed and detail study for the town should have to be carried out by concerned parties.

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